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Recent Research on Post-Frame Building Systems

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Disaster Engineering

Editorial

Wood construction is a diverse field. While the typical 'stick frame' building is considered the norm, many different building methods, including cross-laminated timber (CLT) and structural insulated panel (SIP) construction have been featured in *Wood Design Focus* recently. In this issue of *Wood Design Focus*, the topic is recent research in post -frame buildings. Post-frame buildings originally were developed in the agricultural sector, serving a need for low-cost, efficient buildings after World War II.

ANSI/ASABE S618, "Post Frame Building System Nomenclature" was approved on December 31, 2010. This standard provides a systematic listing of terms associated with post-frame construction. The definition of post-frame buildings is given as:

> *A building characterized by primary structural frames of wood posts as columns and trusses or rafters as roof framing. Roof framing is attached to the posts, either directly or indirectly through girders. Posts are embedded in the soil and supported on isolated footings or are attached to the top of piers, concrete or masonry walls, or slabs-on-grade. Secondary framing members, purlins in the roof, and girts in the walls are attached to the primary framing members to provide lateral support and to transfer sheathing loads, both in-plane and out-of-plane, to the posts and roof framing.*

A copy of the standard is available at http://nfba.org/uploads/Post_Frame_Nomenclature.pdf.

While the roots of post-frame construction extend from the agricultural sector, uses for this building system have grown. Post-frame is limited in size to one– and two– story structures, but post-frame buildings are also used as residences, light commercial, offices, emergency services, and retail spaces. The efficiency of material use - a function of the frames created by the posts and trusses, and the diaphragm action of the roofs - have helped to brand post-frame buildings as a source of green buildings.

The next page demonstrates some images of post-frame buildings taken from ANSI/ASABE S618, courtesy of David Bohnhoff. The articles in this issue of *Wood Design Focus* discuss some of the design challenges specific to post-frame buildings, including the design of shallow post and pier foundations, insulation needs below and around slabs, and a simplified lateral design procedure for predicting the diaphragm and shearwall behavior of post-frame buildings. The National Frame Building Association (www.nfba.org) is the trade association representing the post-frame building industry and provides information on construction, as well as contacts for builders, engineers and suppliers.

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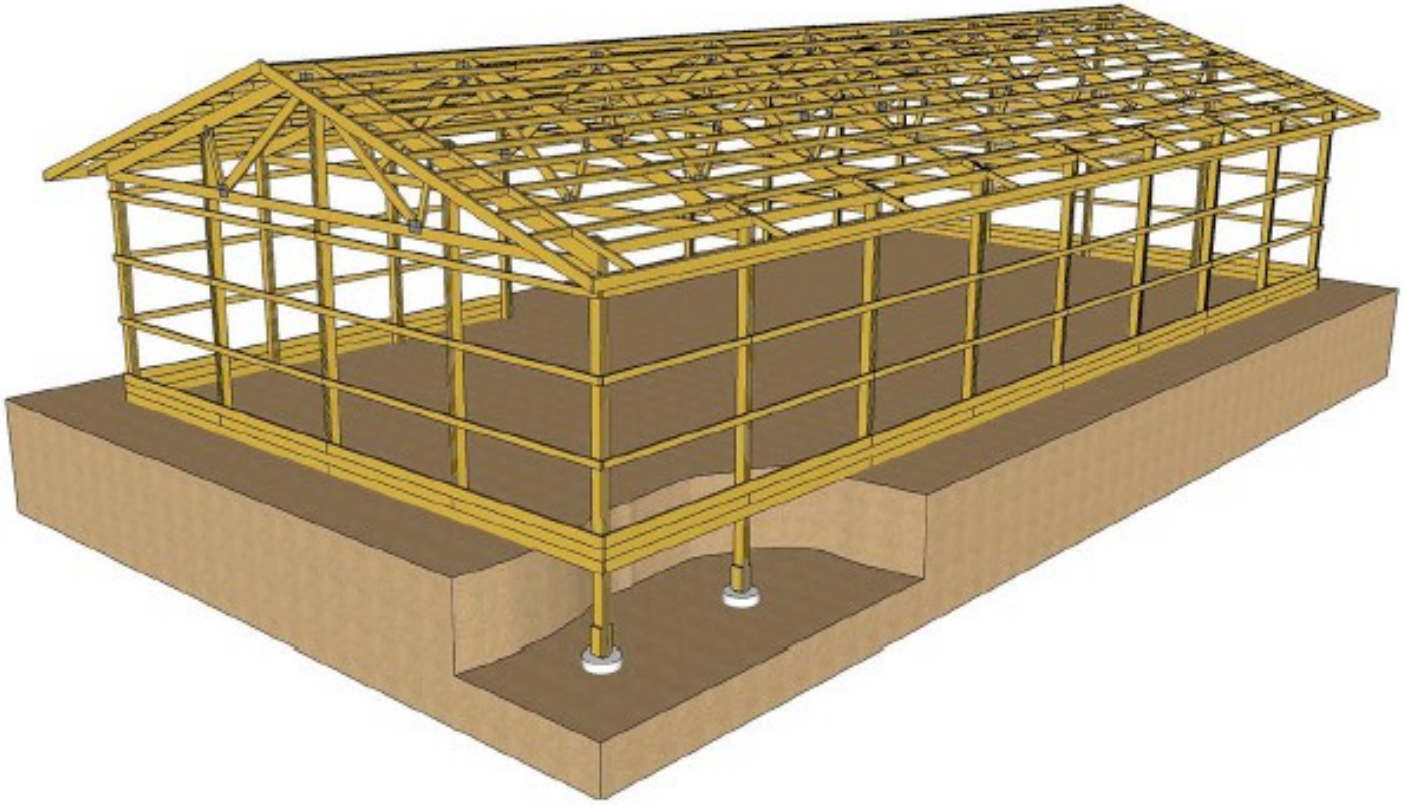

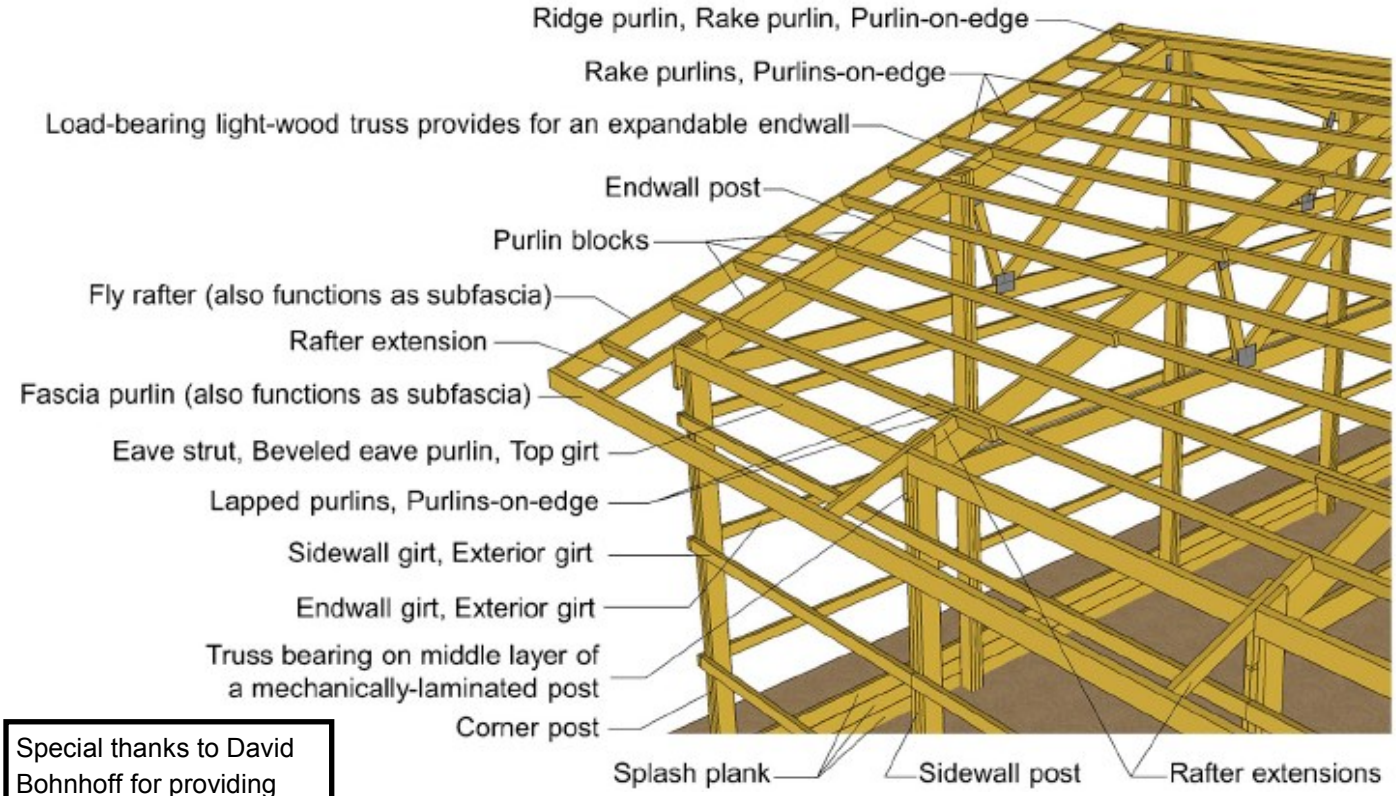

Special thanks to David Bohnhoff for providing these images.

Shallow Post and Pier Foundation Design:
Major Revision of Standard Completed

David R. Bohnhoff, P.E.

In October 2012 the latest version of the American Society of Agricultural and Biological Engineers (ASABE) engineering practice (EP) for the design of shallow post foundations was approved by ANSI. The official designation of this new document is ANSI/ ASAE EP 486.2 *Shallow Post and Pier Foundation Design*, published more than five years after work on the revision began. The lengthy revision process can be attributed to an almost total rewriting of the document, the heart of which are new calculation methods for foundation bearing strength, uplift strength and lateral strength. The revised EP contains 14 main clauses and a commentary. As a way to introduce this revised EP, an overview of each clause follows.

Purpose and Scope

The purpose of ASAE EP486 is to help designers determine the adequacy of shallow, isolated post and pier foundations. This includes ensuring that soil and backfill are not overloaded, foundation elements have adequate strength, frost heave is minimized and lateral movements are not excessive.

This EP contains safety factors and other provisions for allowable stress design (ASD), which is also known as working stress design, and for load and resistance factor design (LRFD), which is also known as strength design. It also contains properties and procedures for modeling soil deformation for use in structural building frame analyses.

Application of the EP is limited to post and pier foundations that (1) have been vertically installed in relatively level terrain, (2) have concentrically loaded footings and (3) have a minimum spacing equal to the greater of 4.5 times the maximum dimension of the post or pier cross-section, or three times the maximum dimension of a footing or attached collar. The third limitation addresses the fact that the shorter the distance between isolated pier/post foundations, the greater the overlap between the “pressure bulbs” surrounding the foundations, and the less applicable will be the equations contained in the EP for estimating maximum uplift, bearing and lateral capacities for isolated pier/post foundations.

This EP applies to piers and posts that are driven into soil, as well as those that are placed into pre-excavated holes and then backfilled. Driven (or displacement) piers consist primarily of steel helical piers (e.g., screw anchors) that are turned into the ground. Driven (or displacement) posts include the short wood posts used to support highway guardrails. Interestingly, helical piers are primarily used to resist bearing and uplift forces, and driven wood posts are primarily used to resist lateral forces.

Normative References

References for documents that are indispensable for the application of the standard are given in Clause 2. This includes six structural design specifications, 10 laboratory soil testing standards, seven in-situ soil testing standards, a preservative-treated wood standard (AWPA U1) and the post-frame building systems nomenclature standard (ANSI/ASABE S618).

Definitions

Clause 3 contains 49 definitions. These are categorized under headings of: foundation types and components; foundation geometry and constraints; material properties and characteristics; and structural loads and analysis.

With respect to foundation types and components, the primary definitions of interest are those for post, pole, pier, post foundation and pier foundation.

A *post* is defined as "a structural column that functions as a major foundation element by providing lateral and vertical support for a structure when it is embedded in the soil. Posts include members of any material with assigned structural properties such as solid or laminated wood, steel, or concrete." A *pole* is simply defined as "a round post."

A *pier* is defined as "a relatively short column partly embedded in the soil to provide lateral and vertical support for a building or other structure. Piers include members of any material with assigned structural properties such as solid or laminated wood, steel, or concrete. Piers differ from embedded posts in that they seldom extend above the lowest horizontal framing element in a structure, and when they do, it is often only a few centimeters."

A *post foundation* is defined as "an assembly consisting of an embedded post and all below-grade elements, which may include a footing, uplift resistance system, and collar." Likewise, a pier foundation is defined as "an assembly consisting of a pier and all below-grade elements, which may include a footing, uplift resistance system, and collar."

Figures 1 to 4 from the revised EP are reproduced here and provide examples of a preservative-treated wood post foundation, a helical pier foundation, a precast concrete pier foundation and a cast-in-place concrete pier foundation, respectively.

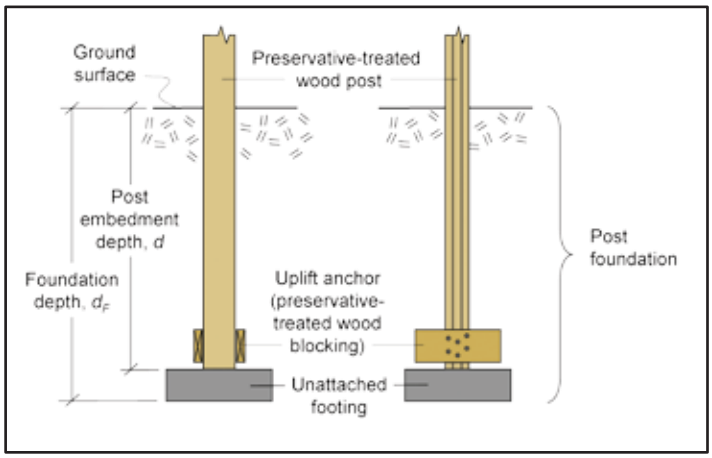

Figure 1. Preservative-Treated Wood Post Foundation

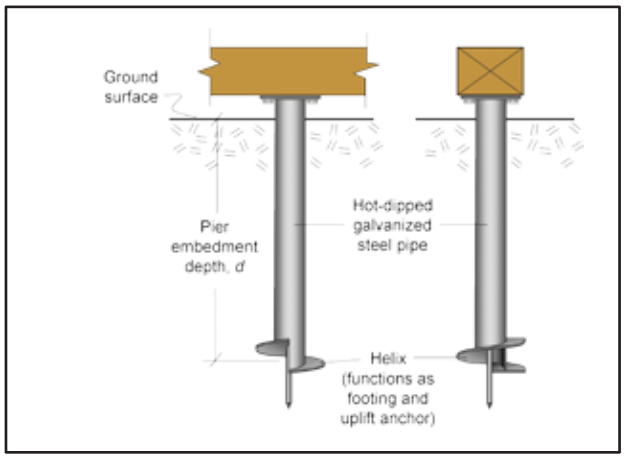

Figure 2. Helical Pier Foundation

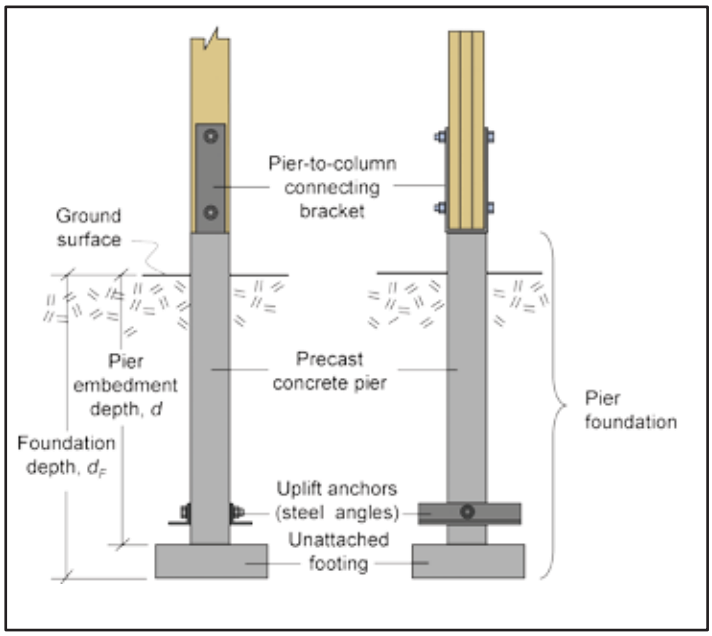

Figure 3. Precast Concrete Pier Foundation

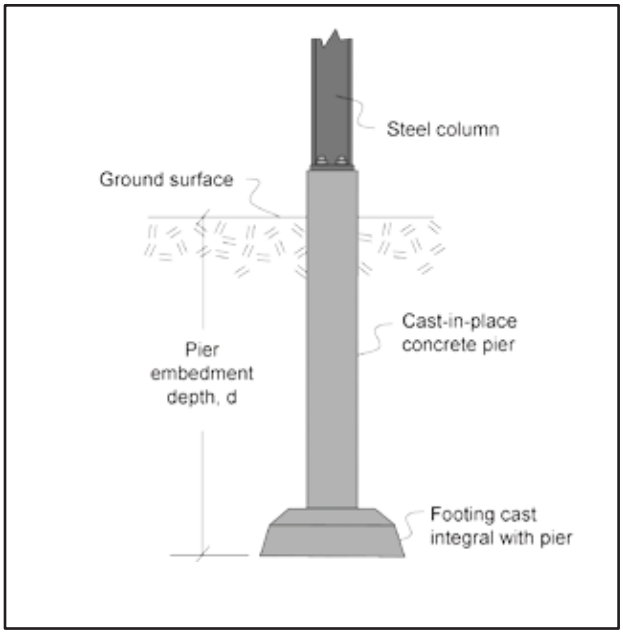

Figure 4. Cast-in-Place Concrete Pier Foundation

Nomenclature

The fourth EP clause contains a list of 110 variables with a symbol, description, and where applicable, a suggested set of units given for each variable. One of the primary objectives when selecting nomenclature was to align verbiage and symbols with those commonly used in geotechnical circles. In two cases, meeting this objective resulted in a switch from what was used in previous versions of the EP.

Soil and Backfill Properties

This clause contains provisions for establishing Young's modulus, undrained shear strength, and friction angle of soils from applicable soil tests. Either laboratory or in-situ testing or a combination of laboratory and in-situ testing can be used to obtain all information needed for post or pier foundation design. Soil tests remove uncertainty associated with the use of presumptive soil properties, and thus lower factors of safety are associated with calculations where soil characteristics have been ascertained through testing.

Clause 5 also addresses soils that should be avoided during post and pier construction. It also addresses appropriate backfill materials, and it contains a table of presumptive soil properties that can be used in the absence of soil test data.

Foundation Material Properties

This clause contains material requirements for post and pier foundation elements, including: minimum concrete compressive strengths; minimum thicknesses and reinforcement requirements for both cast-in-place and precast concrete footings; longitudinal reinforcement, shear reinforcement and concrete cover requirements for concrete piers; and preservative treatment, size and mechanical fastener requirements for embedded wood posts and piers. With respect to precast concrete footings, a thickness as thin as four inches is allowed, provided the footing is placed on a flat compacted base and load-induced forces do not dictate a thicker footing.

As long as the unconfined compressive strength of controlled low-strength material (CLSM) exceeds the ultimate bearing capacity at the base of a post hole, it can be placed between the bottom of a precast concrete (or wood) footing and the underlying soil to increase the effective bearing area of the footing. In lieu of using a CLSM base for footings, some builders have compacted a non-hydrated (i.e., dry) concrete mix in the base of holes. The EP commentary notes that non-hydrated concrete mixes that are compacted within a soil mass and allowed to self-hydrate should obtain unconfined compressive strengths that more than double the 8 MPa limit for classification as a CLSM. Implied by this statement is that the practice of using non-hydrated concrete mixes in this manner is sound.

Structural Load Combinations

Clause 7 contains both ASD and LRFD load combinations from ASCE-7. These load combinations are included in the EP to ensure consistency between soil resistance factors introduced in the EP and the ASCE 7 load factors.

All ASCE-7 nominal loads are included in the EP with the exception that loads due to lateral earth pressure and those due to ground water pressure have not been included. Loads due to lateral earth pressure are not included because soil is treated and modeled as a structural element and not as an applied load (i.e., it is on the resistance side of the equation). Ground water pressure is not included because it is assumed that ground water pressure acts equally on all sides of an embedded post or pier foundation and thus has no net effect on the behavior of embedded elements.

Structural Analysis

Structural analysis is the determination of the forces induced in a post or pier foundation by applied structural loads. Two methods for accomplishing this are outlined in the EP: the Universal Method and the Simplified Method. Alternative methods not covered in the EP are available and may be able to provide more accurate analyses. In all cases, sound engineering judgment should guide selection and application of the design procedure.

The Universal Method can be used to analyze any post or pier foundation. It involves the use of a series of horizontal soil springs to model the interaction between a foundation and the surrounding soil (see Figure 5). The stiffness of an individual spring, K_H, located at depth, z, is given as $K_H = t\,k\,b$ where: t is thickness of the soil layer represented by the spring; b is width of the post or pier, footing, or collar upon which soil represented by the spring is acting; and k is modulus of horizontal subgrade reaction at depth z. The modulus of horizontal subgrade reaction is the ratio of average contact pressure (between foundation and soil) and the horizontal movement of

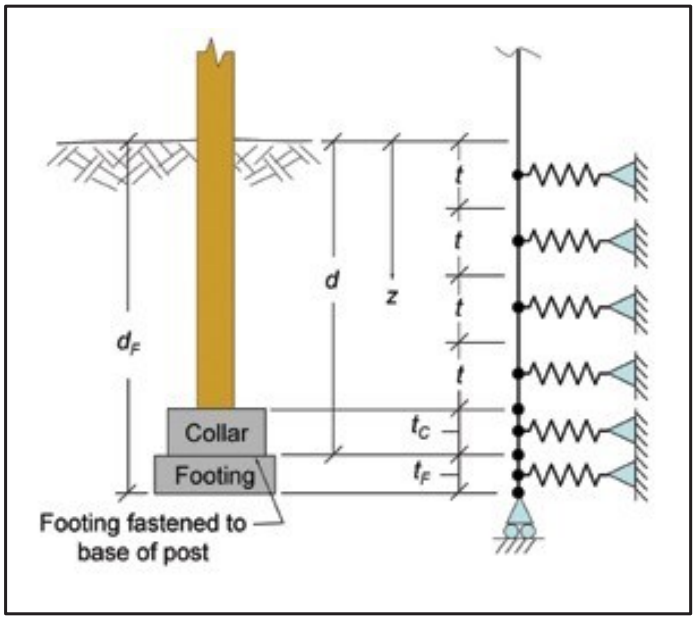

Figure 5. Two-Dimensional Structural Analog For a Post and Pier Foundation.

Different soil springs are used to model soil acting on the collar, attached footing, and post or pier because of the difference in width of the three foundation elements.

the foundation and is equated to two times Young's modulus (at the depth in question) divided by width *b*.

The Simplified Method uses a fixed-based structural analog to determine the bending moment, axial, and shear forces induced in the post or pier near the ground surface. These forces are then substituted in the appropriate equations to determine lateral soil pressures as well as the ground surface displacement and rotation of the post or pier. During the development of these equations it was assumed that (1) at-grade pier and post forces are not dependent on below-grade deformations, (2) the below-grade portion of the foundation has an infinite flexural rigidity, (3) soil is homogeneous for the entire embedment depth, (4) modulus of horizontal subgrade reaction *k* is either constant for all depths below grade or linearly increases with depth below grade, (5) width *b* of the below-grade portion of the foundation is constant (this generally means that there are no attached collars or footings that are effective in resisting lateral soil forces), and (6) groundline shear, V_G, and groundline bending moment, M_G, would not independently cause post rotation in opposite directions.

These assumptions collectively turn a highly indeterminate structural analysis problem into a determinate analysis. The second assumption (i.e., that the post or pier has an infinite bending stiffness) sets a foundation depth limit on use of the Simplified Method. When this depth is exceeded, the Universal Method must be used to calculate lateral soil pressures and foundation forces. There is no depth limit on use of the Universal Method.

The Simplified Method has the advantage that it does not require estimates of soil stiffness or bending stiffness of the post or pier to determine soil forces. However, relative to the Universal Method, the Simplified Method is associated with higher factors of safety to account for the simplifying assumptions associated with its use.

Resistance and Safety Factors

In previous versions of EP486, safety factors were incorporated into presumptive soil properties and design values, and thus designers had no measure of the actual magnitude of the safety factors associated with their designs. In addition, there were no adjustment factors or recommendations to account for more accurate methods of analyses or to enable higher levels of risk in design.

The revised version includes ASD safety factors and LRFD resistance factors. Tabulated safety (and resistance) factors differ depending on the strength property (i.e., lateral, uplift or bearing strength) being calculated, on the test methods used to determine soil properties, and on general soil type (i.e., cohesive versus cohesionless). In addition, safety and resistance factors for lateral strength assessment also depend on whether the Universal or Simplified Method of analysis was used to determine soil pressures, and in the case of cohesionless soils, safety and resistance factors are also a function of soil friction angle.

For buildings and other structures that represent a low risk to human life in the event of a failure, resistance factors may be increased 25 percent (multiplied by 1.25), and safety factors may be reduced 20 percent.

Bearing Strength Assessment

Under previous editions of EP486, bearing strength was exclusively based on presumptive allowable vertical soil pressures. Actual tabulated values were applicable for footings one foot wide and one foot deep. However, it was permissible to increase the tabulated values by 20% for each additional foot of width and/or depth to a maximum of three times the tabulated value.

In the revised EP, bearing strengths are based on

ultimate bearing capacities obtained from in-situ measurements or calculated using the general bearing capacity equation. In-situ measurements that can be used to determine ultimate bearing capacity include the standard penetration test (SPT), the cone penetration test (CPT) and the pressuremeter test (PMT). Correction factors are included in equations for cohesionless soils to account for water table depth relative to foundation depth.

In the end, the methods for determination of foundation bearing strength embodied in the new EP provide more realistic design values than the previous editions, and in most cases, these values will enable assignment of greater bearing strengths to the typical post or pier foundation.

Lateral Strength Assessment

In the revised EP, the lateral strength of pier and post foundations is dictated by the ultimate lateral resisting pressure of the soil, p_U. This resisting pressure can be determined directly from cone penetrometer or PMT data or can be calculated from soil properties (soil friction angle and cohesion) using equations given in the EP. The equations used to calculate p_U from soil properties will provide a p_U that is three times the Rankine passive pressure.

When the Universal Method is used, the designer simply checks that at every spring location p_U is greater than $f_L F_S/(t\,b)$ for ASD (or that p_U is greater than $F_S/(R_L t\ b)$ for LRFD), where f_L is the ASD factor of safety for lateral strength assessment; R_L is the LRFD resistance factor for lateral strength assessment; F_S is the force in the spring at depth z due to the applied structural loads; t is thickness of the soil layer represented by the spring; and b is width of the post or pier, footing, or collar upon which soil represented by the spring is acting.

When the Simplified Method is used, the designer checks that M_U is greater than $f_L M_G$ for ASD (or that M_U is greater than M_G / R_L for LRFD), where M_U is the ultimate moment that can be applied to a post or pier foundation at its groundline without causing a soil failure, and M_G is the moment induced in the post or pier foundation at its groundline by applied structural loads. A series of equations for calculating M_U are compiled in the EP for different soil types and constraint conditions. The manner in which these equations are solved guarantees that $V_U \geq V_G / R_L$ for LRFD and that $V_U \geq f_L V_G$ for ASD, where: V_U is the shear force that can be applied to a post or pier at its groundline without causing a soil failure, and V_G is the shear force induced in the post or pier at groundline by the applied structural loads.

Uplift Strength Assessment

Foundation uplift strength is due to the combination of foundation mass and resistance to uplift provided by soil mass. Attaching a footing, collar, uplift blocking or any other device that effectively enlarges the foundation's base can significantly increase resistance to upward foundation displacement. This resistance is provided by the weight of the soil mass located above the anchorage system plus the resistance to movement of this soil mass.

To move the soil mass located above the anchorage system requires that a failure plane form in the soil. This failure plane extends upward and outward from the edges of the anchorage system. Unlike previous editions of the EP, the revised EP recognizes the fact that this failure plane may or may not reach the ground surface (what actually happens depends on soil properties and the depth and width of the anchorage system). A *shallow foundation under uplift* is a foundation associated with a failure plane that reaches the ground surface. Conversely, a *deep foundation under uplift* is a foundation associated with a failure plane that does not extend to the ground surface. It follows that the first step in uplift calculations is to determine whether a foundation is shallow or deep under uplift. When this has been done, the appropriate EP equation can be used to determine the overall resistance to uplift provided by the soil mass.

In addition to calculation of uplift strength this clause also contains requirements for anchorage system attachment and backfill compaction.

Frost Heave Considerations

An entire clause in the revised EP is dedicated to minimizing the effects of frost heave. This includes recommendations for footing location, water drainage, working with fine-grained soils, concrete backfill and concrete floors.

Installation Requirements

The last clause in the revised EP covers two construction-related factors that can significantly affect structural performance: soil compaction and component placement. In short, all disturbed soil at the base of a hole must be compacted to a

magnitude consistent with the soil bearing capacity assumed in design, and soil upon which a precast concrete footing will be placed must be flat and level. In addition, the installed depth of a post or pier foundation must not be less than 90% of the specified depth. Precast concrete footings must be placed so that the center of the footing is within a distance *b*/2 of the center of the post or pier it supports, where *b* is the width of the post or pier. Cast-in-place concrete footings must be placed so that distance from the center of the post or pier to the nearest edge of the footing is not less than half the specified width of the footing.

Summary

The newly released version of ASAE EP 486 is significantly different from the version it replaces. It contains completely different methods for calculating bearing, lateral and uplift strengths of both pier and post foundations, and unlike previous versions, it contains safety and resistance factors as well as many methods for obtaining soil properties from on-site soil tests. The advantage of on-site soil testing is that it reduces uncertainty in design. The revised EP enables designers to take advantage of this reduced uncertainty with the use of lower factors of safety. It is important to note that the new EP does not require soil testing; it simply enables the use of lower safety factors when and where soil tests have been performed.

Because the revision is so extensive, parts of it are bound to cause confusion among designers as they work through them for the first time or two. An obvious way to clarify some of the new procedures, and thus give designers more confidence in the numbers they generate, is to develop example analyses. This project has been discussed by the NFBA Technical and Research Committee and is something that end users should expect to see in the future.

The EP can be downloaded for a fee from the ASABE Technical Library (https://elibrary.asabe.org/). The price is $38 for ASABE members and $55 for non-ASABE members.

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Below-Grade Insulation for Post-Frame Buildings

Minimizing Frost Heave

David R. Bohnhoff, P.E.

Introduction

It seems that recently more designers are questioning how to best insulate the foundation of a post-frame building. I attribute this to an increase in the number of heated post-frame buildings being constructed, along with an increased emphasis on reducing heat loss/gain in these buildings.

The latter is fueled by the green building movement and corresponding changes in energy conservation codes.

In virtually all cases where a post-frame building foundation is being insulated, the building has a concrete slab. The questions I get generally come from designers who have seen several different systems used to insulate these slabs, including systems that utilize exterior horizontal wing insulation (Figure 1a), systems that feature only vertical exterior insulation (Figure 1b), and systems in which insulation is placed under the concrete slab (Figures 1c and 1d).

Two Design Goals

It's important to understand that there are two principal reasons for installing below-grade insulation. The first is to control building heat loss/gain in an effort to minimize building operating costs and to reduce consumption of non-renewable natural resources associated with energy production. The second is to prevent frost from heaving a slab and causing structural damage. The latter is a concern only in colder regions with frost-susceptible soils.

Designs for control of building heat loss/gain are based largely on requirements in American National Standards Institute (ANSI) and American Society of Heating, Refrigerating and Air-Conditioning Engineers (ASHRAE) Standard 90.1-2007 *Energy Standard for Buildings Except Low- Rise Residential Buildings*.

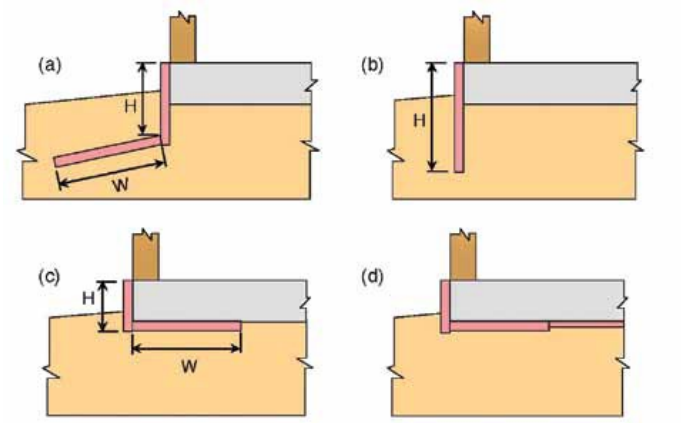

Figure 1. Below-Grade Insulation Options for a Concrete Slab-on-Grade.

(a) vertical and horizontal wing insulation, (b) vertical insulation only, (c) insulation on outside and underside of perimeter edge, and (d) insulation on the outside edge and entire underside of slab.

Designs for minimization of damage due to frost heave are based largely on requirements in Structural Engineering Institute (SEI) and American Society of Civil Engineering (ASCE) 32-01 *Design and Construction of Frost-Protected Shallow Foundations*. The fact that SEI and ASCE administer the standard that addresses frost heave and ASHRAE administers the standard that addresses building heat loss/gain underscores the two distinctly different purposes for using below-grade insulation - one structural and one energy related.

This article is the first in a two-part series on below-grade insulation of post-frame buildings. As the title indicates, this first article is dedicated to building design for frost-heave control. The second article will cover design requirements for heat-transfer control, as well as design details for below-grade insulation of post-frame buildings with embedded posts. These design details will be accompanied by a discussion on their constructability.

The Cause of Frost Heave

In areas where average daily temperatures stay below freezing for extended periods of time, soil heaving due to ice segregation can be a major concern. *Ice segregation* is the formation of discrete ice layers or lenses within the soil due to the migration and subsequent freezing of pore water, which is water in the spaces between soil particles. *Frost heaving* (also known as *soil heaving* or *frost action*) directly results from the fact that water expands approximately 9% in volume when it freezes.

The temperature at which pore water freezes depends largely on solute concentrations. Pore water with low solute concentrations will freeze within a fraction of a degree of 32ºF, whereas pore water with a high solute concentration may not completely freeze until its temperature has dropped to 25ºF.

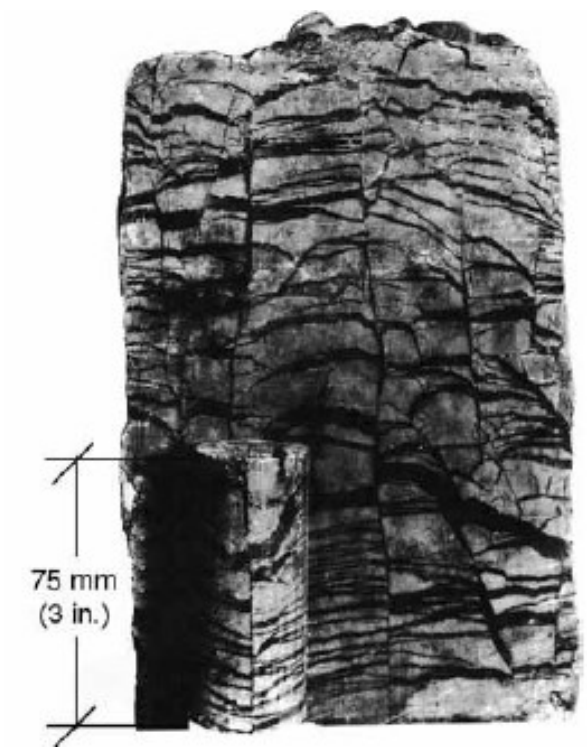

Figure 2. Sample of Frozen Clay Showing Ice Segregation (*Canadian Foundation Engineering Manual*)

If pore water present near the soil surface at the beginning of winter were the only water to turn to ice, there would be no real frost heaving issues. Large ice layers and lenses (and hence problems) result when pore water turns to ice and then sucks water from warmer areas by capillary action. The suction that ice exerts on warmer soil water is termed *cryosuction*. As cryosuction feeds capillary water to the underside of ice layers and lenses, their thickness grows. The term *ice segregation* is used to describe this ice formation action because it segregates regions of previously mixed soil and water into regions of ice and dry soil. Segregation ice often forms regularly spaced layers as shown in Figure 2. As each layer forms, it tends to suck the soil beneath it dry. When the force of cryosuction is no longer able to lift water from below, thickening of the current layer ceases, and cooling proceeds downward until a new ice layer can begin to form at a greater depth.

It is important to understand that ice segregation (and hence soil heaving) requires the presence of three components: frost-susceptible soils, water, and freezing temperatures. Remove any one of these three, and frost heave does not occur.

Frost Susceptibility of Soils

The frost susceptibility of a soil is largely a function of the amount and relative size of smaller soil particles. Smaller particles fill spaces between larger particles, thus reducing the effective size of soil pores. The smaller the effective pore size, the greater the capillary action within the soil.

In the *Canadian Foundation Engineering Manual*, soil scientist Arthur Casagrande reports that "under natural conditions and with sufficient water supply, expect considerable ice segregation in uniform soils containing more than 3% of grains smaller than 0.008 inches and in very uniform soils containing more than 10% smaller than 0.0008 inches. No ice segregation was observed in soil containing less than 1% of grains smaller than 0.0008 inches, even if the groundwater was as high as the frost line." The manual also states that "the borderline between soils that are frost-susceptible and those that are not is not distinct, and those which appear to fall just clear of the Casagrande criteria should be treated with caution."

Table 1 lists the frost susceptibility of soils. To relate soil types in the table to Casagrande's limits, note that clay-sized particles are defined as those less than 0.00008 inches, silt-sized particles as those between 0.00008 and 0.003 inches, and sand-sized particles as those between 0.003 and 0.08 inches. As a point of reference, particles less than 0.003 inches in diameter (silts and clays) cannot be distinguished with the naked eye.

Table 1 shows that frost heave is a non-issue when one is building on sands and gravels that do not contain silts and clays. As silt content increases, frost heave becomes more problematic. The most frost-susceptible soils are silts with a low plasticity index (PI). The PI indicates the breadth of the range of soil moisture content values for which a soil exhibits plastic properties. Soils with a high PI tend to have more clay-sized particles and clay-type minerals. On the basis of the previous discussion, one may conclude that soils with a higher PI are more susceptible to frost heave. This is true to a point. As the clay content of a soil increases, a point is reached where the clay content is so high, and effective pore size so small, that water is essentially blocked from moving through the pores. This is why a pure clay soil (i.e., a soil with a very high PI) is not as susceptible to frost heave as a pure silt soil (i.e., a soil with a lower PI).

Frost Penetration Depth

Frost heave is a problem only if the soil under a foundation freezes. To what extent frost penetrates a soil depends on soil type and cover (Figure 3) and on the temperature and duration of the winter weather.

The best measure of overall coldness and duration of winter weather is the air freeze index (AFI). This predictor of frost penetration depth is determined from cumulative freezing degree days. One freezing degree day accumulates for each degree the average daily temperature is below 32ºF, with the average daily temperature taken as the average of the minimum and maximum daily temperatures. Table 2 shows how freezing degree days would accumulate for a 7-day period.

The AFI for a given winter is the largest difference between the maximum freezing degree day cumulative total reached at the start of the winter season and the minimum total reached during the

Table 1. Frost Susceptibility of Soils

Soil Type	Frost Susceptibility
Gravels and Sands	None to Low
Silty and Clayey Gravels	Low to Medium
Silty and Clayey Sands	Low to High
Clays with a High Plasticity Index	Medium
Silts with a High Plasticity Index and	Medium to High
Silts with Low Plasticity Index	Medium to Very High

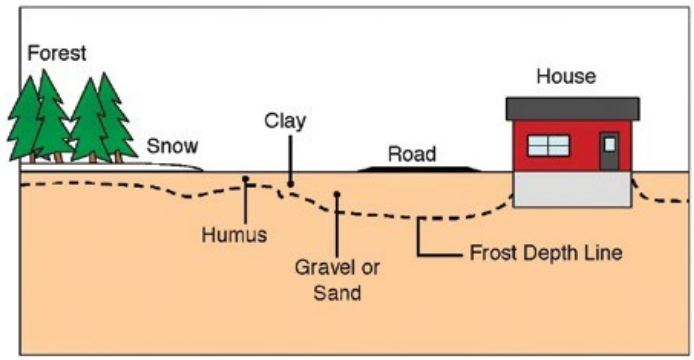

Figure 3. Effect of Soil Cover and Soil Type on Frost Penetration Depth

Figure 4. Air Freezing Index (AFI) Values for a 100-year Mean Return Project

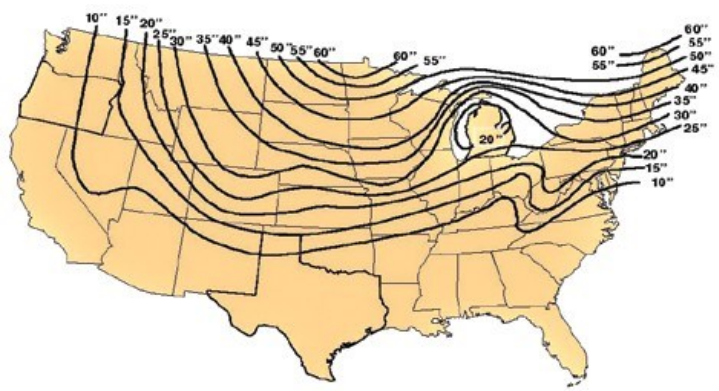

Figure 5. Average Annual Frost Penetration Depths

Table 2. Example of Freezing Degree Day Calculation

Day	Maximum Daily Temperature, °F	Minimum Daily Temperature, °F	Average Daily Temperature, °F	Freezing Degree Days per Day[a], °F•day	Cumulative Freezing Degree Days per Day[b], °F•day
1	23	12	17.5	14.5	14.5
2	19	13	16	16	30.5
3	27	16	21.5	10.5	41
4	33	29	31	1	42
5	39	31	35	-3	39
6	37	24	30.5	1.5	40.5
7	19	8	13.5	18.5	59

[a] A negative sign indicates a day when the average daily temperature was above 32°F and thus more thawing than freezing occurred.
[b] Assume Day 1 start of freezing season.

same winter. Figure 4 contains a map of the AFI_{100} values for the contiguous United States. AFI_{100} values are AFI values with a mean reoccurrence interval of 100 years. In Alaska, AFI_{100} values range from a low of 1,000 ºF•days to a high of 12,000 ºF•days!

Average annual frost penetration depths for the contiguous United States are shown in Figure 5. Because frost frequently penetrates depths greater than the average annual depth, foundations on frost-susceptible soils should always be placed at depths greater than those in Figure 5 unless other steps are taken to keep the soil below the foundation dry or from freezing.

Frost Heave of Shallow Foundations

With respect to frost heave, a *shallow foundation* is defined as any foundation whose base is not located below frost penetration depth. According to this definition, the embedded post-foundations used to support many buildings would not be classified as shallow foundations because they are purposely designed and installed with their base below frost penetration depth.

A concrete slab-on-grade floor placed inside a building with embedded posts should be treated as a shallow foundation. When overlying frost-susceptible soils, this slab may be subjected to frost heave unless the soil is kept relatively dry or is prevented from freezing. Frost heave in this case results in differential movement between the slab and embedded posts and can result in structural damage. Heaving will tend to be more uneven in a heated building because heat loss through the slab is more likely to keep soil under interior portions of the slab from freezing, while soil under slab edges freezes and heaves. It is not uncommon for such action to result in the formation of a crack in the slab that runs parallel to an exterior wall. This same differential slab movement and associated cracking are also a concern in buildings with posts mounted on the slab.

In general, if moisture in frost-susceptible soils underlying a slab is allowed to freeze, the slab would need be engineered with a proper amount of steel reinforcing to enable the slab to span, without failure, from high point to high point on the swollen soil surface. However, because no quick or accurate method currently exists for predicting variations in the location and magnitude of frost heave under a slab, designing a slab to float on frozen soil is really not an option available to the designer.

Another reason for not allowing soil under a slab to freeze is that frozen soil becomes extremely weak as it thaws. In fact, soil left saturated by melting water is generally measurably weaker than it was in its prefreeze state. This reduction in soil strength is referred to as thaw-weakening. It is a major problem with clay and silt-type soils and is largely responsible for the formation of potholes in asphalt pavements.

Documents containing procedures for sizing concrete slab-on-grade floors include these: American Concrete Institute (ACI) 360R-06 *Design of Concrete Slabs on Ground*, the U.S. Department of Defense's Unified Facilities Criteria (UFC) 3-320-06A *Concrete Floor Slabs on Grade Subjected to Heavy Loads* (available for free at www.wbdg.org/ccb/DOD/UFC/ufc_3_320_06a.pdf), *Concrete Floors on Ground,* written by J. A. Farny and M. Tarr and published by the Portland Cement Association (PCA), and *Design of Post-Tensioned Slabs -on-Ground,* published by the Post-Tensioning Institute (PTI). Although none of these documents contains methodology for predicting the effects of frost heave,

the PTI document contains procedures for determining differential slab heave due to moisture content changes in expansive and compressible soils.

Methods for Minimizing Frost Heave Under Shallow Foundations

To minimize frost heave potential, a designer has two primary options: place the foundation on well-drained soils that are not susceptible to frost heave, or prevent soil under the foundation from freezing.

Building on well-drained soils that are not frost heave susceptible is the most common method used to minimize frost heave. This option frequently involves replacing any frost-susceptible soils located within 1 or 2 feet of grade with sands and gravels. The depth to which such soil replacement is needed depends primarily on frost penetration depth and on the quantity of heat lost from the building to the soil.

It is always beneficial to construct the subgrade so that the soil underlying the foundation stays relatively dry. Not only is this important for minimization of frost heave, it is also important for maintaining soil bearing capacity, maximizing soil thermal resistance, minimizing liquid and vapor diffusion into a building, and extending the durability of building components in soil contact.

The extent to which the soil underlying a foundation can be kept dry depends on such factors as foundation elevation relative to the surrounding terrain, soil type, distance to the ground water table, and type of perimeter drainage system (if employed). With respect to soil type, coarse-grained soils (i.e., sands and gravels) are much preferred because they drain quickly and are associated with minimal capillary action. As a rough approximation, capillary rise is 3 inches in a fine gravel, 6 inches in a coarse sand, 10 inches in a medium sand, 20 inches in a fine sand, 40 inches in a silt, and 80 inches in a clay. As previously noted, capillary action for a particular soil is largely a function of the amount and relative size of the smaller particles in the soil as they dictate capillary size. Thus, a soil that is 80% sand and 20% clay is likely to have a capillary rise approaching that of a pure clay soil.

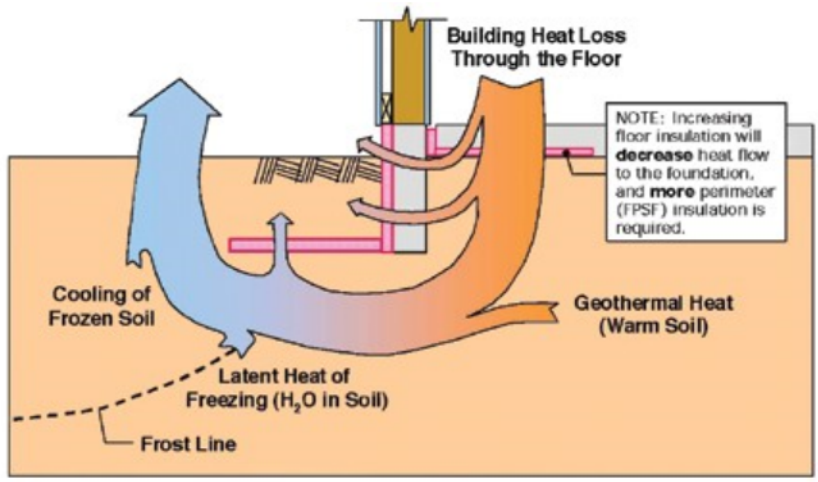

Figure 6. FPSF Heat Flow Diagram for a Heated Building with Optional Floor Insulation

The second option for control of frost heave is to prevent freezing of soils located under the slab with a strategic use of insulation as shown in Figure 6. This insulation keeps building-heat energy and/or heat energy stored in deep underlying soil from being rapidly lost to cool winter air. When properly sized and placed, the insulation keeps all soil located immediately below even the shallowest of foundations from freezing. Shallow foundations that feature such insulation systems are commonly referred to as *frost-protected shallow foundations* (FPSFs).

SEI/ASCE32-01

Procedures for sizing and locating FPSF insulation are contained in SEI/ASCE 32-01 *Design and Construction of Frost-Protected Shallow Foundations*. SEI/ASCE 32-01 is actually a slightly modified version of a National Association of Home Builders Research Center (NAHBRC) document published in 1994. The NAHBRC document was based on Scandinavian codes in existence since the 1960s; since the early 1950s, more than 1.5 million homes have been built with FPSFs in Sweden alone. NAHBRC prescriptive requirements for FPSFs appeared in the original version of the International Residential Code published in 2000. SEI/ASCE 32-01 was published in 2001 (as the -01 indicates).

The manner in which FPSF insulation is installed depends on the thermal classification of the building. SEI/ASCE 32-01 defines three building types: heated, unheated, and semiheated. A *heated* building is one with a minimum average monthly indoor temperature greater than 63ºF. An

Table 3. Minimum Insulation Requirements for Frost-Protected Footings in Heated Buildings[a]

AFI_{100}[b], °F•days	Vertical Insulation		Horizontal Wing Insulation Along Walls		Horizontal Wing Insulation At Corners		
	Minimum R-Value[c], h•ft²•°F/Btu	Minimum Depth Below Grade, *D*, inches	Minimum R-Value[c], h•ft²•°F/Btu	Width, W_w, inches	Minimum R-Value[c], h•ft²•°F/Btu	Width, W_c, inches	Length, L_c, inches
1500 or less	4.5	12	NR	NR	NR	NR	NR
2000	5.6	14	NR	NR	NR	NR	NR
2500	6.7	16	1.7	12	4.9	24	40
3000	7.8	16	6.5	12	8.6	24	40
3500	9.0	16	8.0	24	11.2	30	60
4000	10.1	16	10.5	24	13.1	36	60
4500	12.0	16	12.0	36	15.0	48	80

[a] Insulation requirements are for protection against frost damage in heated buildings. Greater values may be required to meet energy conservation standards. Interpolation between values is permissible.
[b] Air freezing index for 100-year mean return period. See Figure 4 for values by location.
[c] Use the approximate *effective* R-value from Table 4 when determining whether an insulation meets the minimum required R-value.

unheated building is one with a minimum average monthly indoor temperature less than 41ºF. Buildings falling between these two extremes are defined as *semiheated*.

Slab-on-grade floors of *heated* buildings are protected from frost heave by using the insulation systems shown in Figures 1a and 1b. Although systems with insulation located under the slab (Figures 1c and 1d) prevent building heat loss, it may not prevent soil located directly under the slab from freezing at locations near the building perimeter.

Simplified FPSF Design Method for Heated Buildings

SEI/ASCE 32-01 contains both a simplified and a detailed method for determining insulation requirements for heated buildings. The simplified FPSF design method is a prescriptive specification requiring insulation in accordance with Table 3. This table contains R-values and dimensions for exterior vertical insulation and exterior horizontal wing insulation. Dimensions used in Table 3 are graphically defined in Figure 7. Table 4 contains effective insulation R-values to be used when one is determining whether insulation meets the required minimum R-value specified in Table 3.

The simplified design method cannot be used when there is insulation underlying the slab with an R-value greater than 10 h•ft²•°F/Btu.

Detailed FPSF Design Method for Heated Buildings

In reality, there are an infinite number of combinations of insulation dimensions and R-values for both vertical and horizontal wing insulation that can be used to protect a heated building from frost heave. The SEI/ASCE 32-01 detailed FPSF design method is a performance specification that provides the designer with the flexibility to select the combination of insulation R-values and dimensions that work best for the job. In addition, the detailed method enables the designer to account for effects of insulation placed directly under a slab-on-grade floor. As noted in Figure 6, such insulation will decrease heat flow to the foundation, thereby requiring more perimeter insulation.

The detailed FPSF design method consists of the following steps:

Step 1: Determine the site's design AFI. Approximate the AFI from Figure 4 or obtain a more site-specific value from the National Climatic Data Center FPSF Web site (www.ncdc.noaa.gov/oa/fpsf/).

Step 2: Calculate the R-value for the floor slab, R_F. Consider all insulating materials in the cross-section, including any floor covering. Table 5 contains R-values for selected materials. When determining R_F, use dry-condition R-values for all materials including insulation. If R_F varies over the slab area, calculate R_F as the average over the perimeter (i.e., outer) 3 feet of the floor. If R_F exceeds 28 h•ft²•ºF/Btu,

Table 4. Design Values for FPSF Insulation Materials

ASTM C578 Insulation Type[a]		ASTM C578 Minimum Insulation Density, lbm/ft^3	Effective R-value per Inch of Thickness, R_{eff}[b], (h•ft^2•°F/Btu)/in		ASTM C578 Nominal R-value per inch of Thickness, (h•ft^2•°F/Btu)/in	ASTM C578 Compression Strength at Yield or 10% Deformation[a], lbf/in^2	Allowable Bearing Capacity	Minimum Insulation Thickness, Inches	
			Vertical	Horizontal				Vertical	Horizontal
Expanded Polystyrene	II	1.35	3.2	2.6	4.0	15	720	2	2
	IX	1.9	3.4	2.8	4.2	25	1,200	1.5	2
Extruded Polystyrene (XPS)	X	1.35	4.5	4.0	5.0	15	720	1.5	2
	IV	1.6	4.5	4.0	5.0	25	1,200	1	1.5
	VI	1.8	4.5	4.0	5.0	40	1,920	1	1
	VII	2.2	4.5	4.0	5.0	60	2,880	1	1
	V	3.0	4.5	4.0	5.0	100	4,800	1	1

[a] The primary means for identifying ASTM C578 insulation type at a retail level is via the compression strength at yield or 10% deformation. For example, Owens Corning's FoamulaR 150, 250, 400, 600 and 1000 are XPS insulations with compressive strengths of 15.0, 25.0, 40.0, 60.0, and 100 lbf/in^2, respectively, and they are classified as ASTM C578 Types X, IV, VI, VII and V, respectively.
[b] Effective R-values are based on laboratory tests and field studies of insulation products under long-term exposure to moist, below-ground conditions. 'Vertical' effective R-values shall be used for insulation placed vertically on exterior foundation walls. 'Horizontal' effective R-values shall be used for insulation placed horizontally below ground.
[c] Allowable bearing capacity is based on ASTM C578 compressive strength at 10% deformation divided by a safety factor of 3.0 for conditions without cyclic loading (e.g, highway vehicle loading).

Table 5. Thermal Properties of Selected Floor Materials (ASHRAE Handbook of Fundamentals)

Material Description	R-value per Inch of Thickness, (h•ft^2•°F/Btu)/in	R-value, h•ft^2•°F/Btu
Plywood or wood subfloor	1.25	
Wood (lumber)	0.90	
Concrete (150 lbm/ft^3)	0.05	
Concrete (100 lbm/ft^3)	0.18	
Gypsum-fiber concrete (87.5% gypsum, 12.5% wood chips)	0.60	
Cement/lime, mortar, and stucco (120 lbm/ft^3)	0.10	
Cement/lime, mortar, and stucco (80 lbm/ft^3)	0.22	
6 mil Plastic		Negligible
Carpet and Fibrous Pad		2.08
Carpet and Rubber Pad		1.23
Cork Tile (0.125 inches thick)		0.28
Terrazzo (1.0 inches thick)		0.08
Tile (asphalt, linoleum, vinyl, rubber)		0.05
Wood Flooring (0.75 inches thick)		0.68

the designer must follow guidelines in the following section for an unheated building, because heat from the building is substantially blocked from moving into the ground and protecting the foundation.

Step 3: Use Table 6 and the information from Steps 1 and 2 to determine the required R-value of the vertical insulation, R_V.

Step 4: Based on the required R_V from Step 3, select an adequate thickness of insulation using the effective R-values, r_{eff}, listed in Table 4. Individual panel thickness shall not be less than the minimum insulation thicknesses listed in the right columns of Table 4. Vertical insulation must extend from a depth *D* to the exterior, above-grade wall without exposing the foundation wall or other thermally conductive materials, as shown in Figure 7.

Step 5: Use Table 7 to select insulation dimensions for situations in which no wing insulation is desired, or where wing insulation is desired at corners only. Note that this wing insulation must have an R-value of 5.7 h•ft^2•°F/Btu. Alternatively, use Tables 8 and 9 to determine wing insulation dimensions and R-values for applications where the depth *D* of all vertical insulation (i.e., that along the wall and at corners) will be fixed at 16 inches.

Step 6: Select an adequate thickness for required wing insulation by dividing the required minimum R-value of the insulation by its effective horizontal R-values, r_{eff}, listed in Table 4.

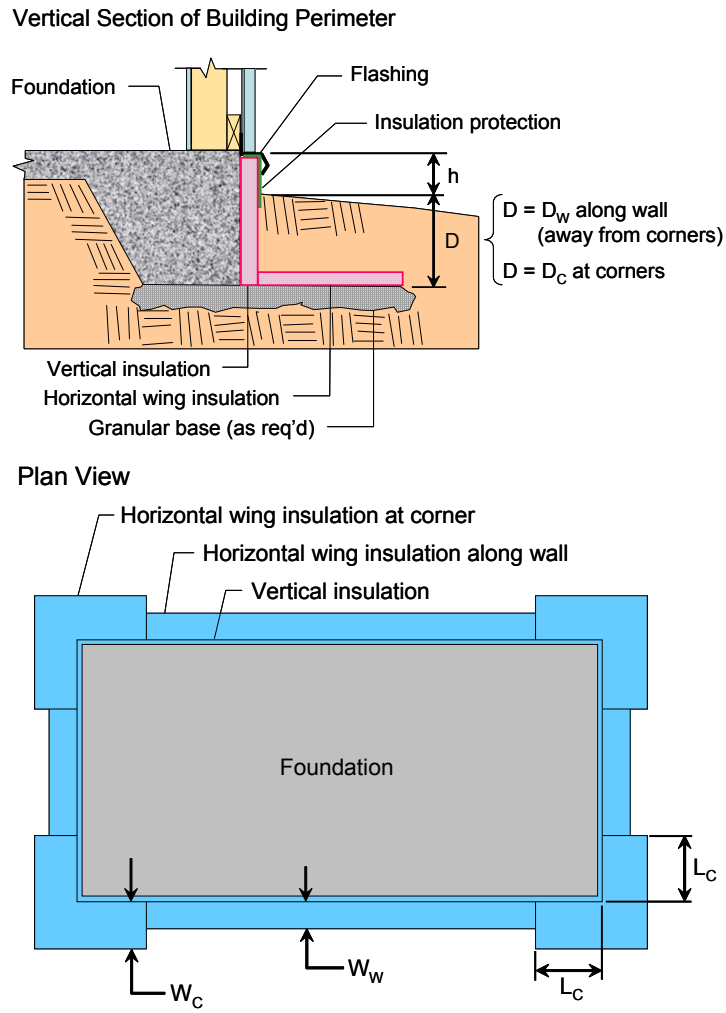

Figure 7. Frost Protected Shallow Foundation Dimensions.

The floor height above the finished grade (dimension *h*) is limited to a maximum of 12 inches when the simplified FPSF design method is used.

Table 6. Required Thermal Resistance of Vertical Wall Insulation

AFI_{100}, °F•days	Minimum R-value of Vertical Wall Insulation[a], R_V, h•ft^2•°F/Btu					
	Floor Height Above Finished Grade, *h*<12 in.			Floor Height Above Finished Grade, *h*=24 in.		
	$R_F \leq 6.0$	$R_F = 15.0$	$R_F = 28.0$	$R_F \leq 6.0$	$R_F = 15.0$	$R_F = 28.0$
375 or less	0.0	4.5	5.7	3.0	5.7	8.5
750	3.0	5.7	8.5	4.6	5.7	11.4
1,500	4.5	5.7	8.5	5.7	5.7	11.4
2,250	5.7	5.7	8.5	5.7	7.4	14.2
3,000	5.7	6.8	9.7	5.7	8.5	15.3
3,750	5.7	8.0	11.4	6.8	9.7	17.0
4,500	6.8	10.2	13.6	8.0	11.9	19.3

[a] Interpolation of R_V values is permitted for *h* values between 12 and 24 inches, R_F values between 6, 15 and 28 h•ft^2•°F/Btu, and all listed AFI_{100} values.

Table 7. Minimum Insulation Dimensions with No Wing Insulation or with Wing Insulation at Corners Only[a]

AFI_{100}, °F•days	No Wing Insulation			Wing Insulation (R=5.7 h•ft^2•°F/Btu) at Corners Only		
	Vertical Insulation Depth Along Walls, D_w, *inches*	Vertical Insulation Depth at Corners, D_C, inches	Vertical Insulation Length at Corners, L_C, inches	Vertical Insulation Depth Along Walls and Corners, D_W and D_C, inches	Vertical Insulation Length at Corners, L_C, inches	Width of Wing Insulation at Corners, W_C, inches
1500 or less	12	12	—	12	—	—
2250	14	14	—	14	—	—
2625	16	24	40	16	40	20
3000	20	32	40	20	40	20
3375	24	40	60	24	60	20
3750	30	51	60	30	60	24
4125	36	63	60	36	60	32
4500	43	71	80	43	80	32

[a] Interpolation of dimensions is permitted for intermediate AFI_{100} values.

Table 8. Required Thermal Resistance of Wing Insulation for Use Along Walls with D_W = 16 Inches[a]

AFI_{100}, °F•days	Minimum R-value of Wing Insulation Along Wall, R_H, h•ft^2•°F/Btu, Per Width of Wing Insulation Along Wall, W_W						
	12 Inches	18 Inches	24 Inches	30 Inches	36 Inches	42 Inches	48 Inches
2250 or less	0.0						
2625	2.5						
3000	6.5	6.1	5.3	4.5			
3375		8.2	7.4	6.5			
3750			9.1	8.5	7.7		
4125			11.2	10.2	9.6	8.9	
4500				12.3	11.4	10.7	10.0

[a] Interpolation of R-values is permitted between W_W values and between AFI_{100} values.

Table 9. Required Thermal Resistance of Wing Insulation for Use at Corners with D_C = 16 inches[a]

AFI_{100}, °F•days	Vertical Insulation Length at Corners, L_C, inches	Minimum R-value of Wing Insulation Along Wall, R_H, h•ft^2•°F/Btu, Per Width of Wing Insulation Along Wall, W_W					
		16 Inches	24 Inches	30 Inches	36 Inches	42 Inches	48 Inches
2250 or less	0	0.0					
2625	40	6.5	4.9	4.0			
3000	40	9.6	8.6	8.0	7.4		
3375	60		11.1	10.5	9.8	9.1	
3750	60		13.1	12.5	12.0	11.2	10.8
4125	60			14.5	13.7	13.0	12.5
4500	80				15.9	15.1	14.8

[a] Interpolation of R-values is permitted between W_W values and between AFI_{100} values.

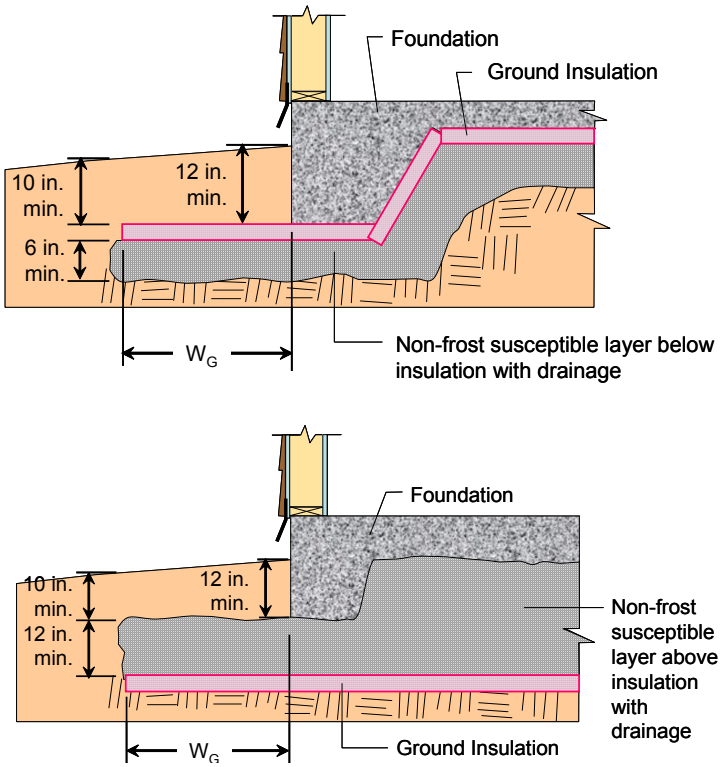

Figure 8. FPSF Dimensions for an Unheated Building.

FPSF Design Method for Unheated Buildings

Geothermal energy is solely relied upon to keep frost-susceptible soils beneath the foundation of unheated buildings from freezing. As Figure 8 shows, this energy is prevented from rapidly leaving the soil by a continuous layer of insulation placed under the entire foundation. A non-frost-susceptible soil layer at least 6 inches thick must be placed below the insulation, or a non-frost-susceptible soil layer at least 12 inches thick must be placed above the insulation. The ground insulation layer and the non-frost-susceptible layer are additive to the minimum footing depth of 12 inches shown in Figure 8. Outside the foundation perimeter, the insulation must have a soil cover at least 10 inches thick.

The distance W_G that the ground insulation is required to extend past the perimeter of the building and the R-value of the insulation are obtained from Table 10. These values are dependent on AFI_{100} values as well as the mean annual temperature for the building site. See Figure 9 for mean annual temperatures. R-values from Table 10 may be reduced by 0.3 h•ft^2•°F/Btu for every 1 inch that the non-frost-susceptible layer thickness is increased beyond the required minimum. In addition, the R-value can be reduced by 0.3 h•ft^2•°F/Btu for every 1 inch increase in soil cover thickness above the 10 inch minimum Finally, W_G may be reduced by 1.25 inches for every 1 inch that the insulation is buried beyond the 10 inch minimum cover.

FPSF design method for semiheated buildings The foundation of buildings classified as semiheated shall be designed in accordance with the detailed FPSF design method for heated buildings, but with the minimum vertical insulation depth increased by 8 inches for both wall and corner areas of the semiheated building.

Table 10. Required Ground Insulation Properties for Unheated Buildings

AFI_{100}, °F•days	Minimum Horizontal Extension[b], W_G, inches	Minimum R-value of Below Slab Insulation and Horizontal Insulation Extensions, Per Mean Annual Temperature				
		≤ 32°F	36°F	38°F	40°F	≥ 41°F
750 or less	30	5.7	5.7	5.7	5.7	5.7
1500	49	13.1	9.7	8.5	8.0	6.8
2250	63	19.4	15.9	13.6	11.4	10.2
3000	79	25.0	21.0	18.2	15.3	14.2
3750	91	31.2	26.1	22.7	-	-
4500	108	37.5	31.8	-	-	-

[a] Interpolation of R-values is permitted between main annual temperature and between AFI_{100} values.
[b] Horizontal extension width is the same for corner locations as it is for points along the wall.

Summary

Two good options exist for minimizing frost heave of shallow foundations in cold climates. First, the foundation can be located on soils not susceptible to frost heave. In this case, not all soils below a foundation need to be non-frost-susceptible, only those located above the frost penetration depth. The second option is to install insulation that prevents soil underlying the slab from freezing. Data and procedures presented in this article can be used to size these insulation systems. In heated buildings, this insulation is located around the outside of the foundation. In unheated buildings, it is located directly under the entire foundation.

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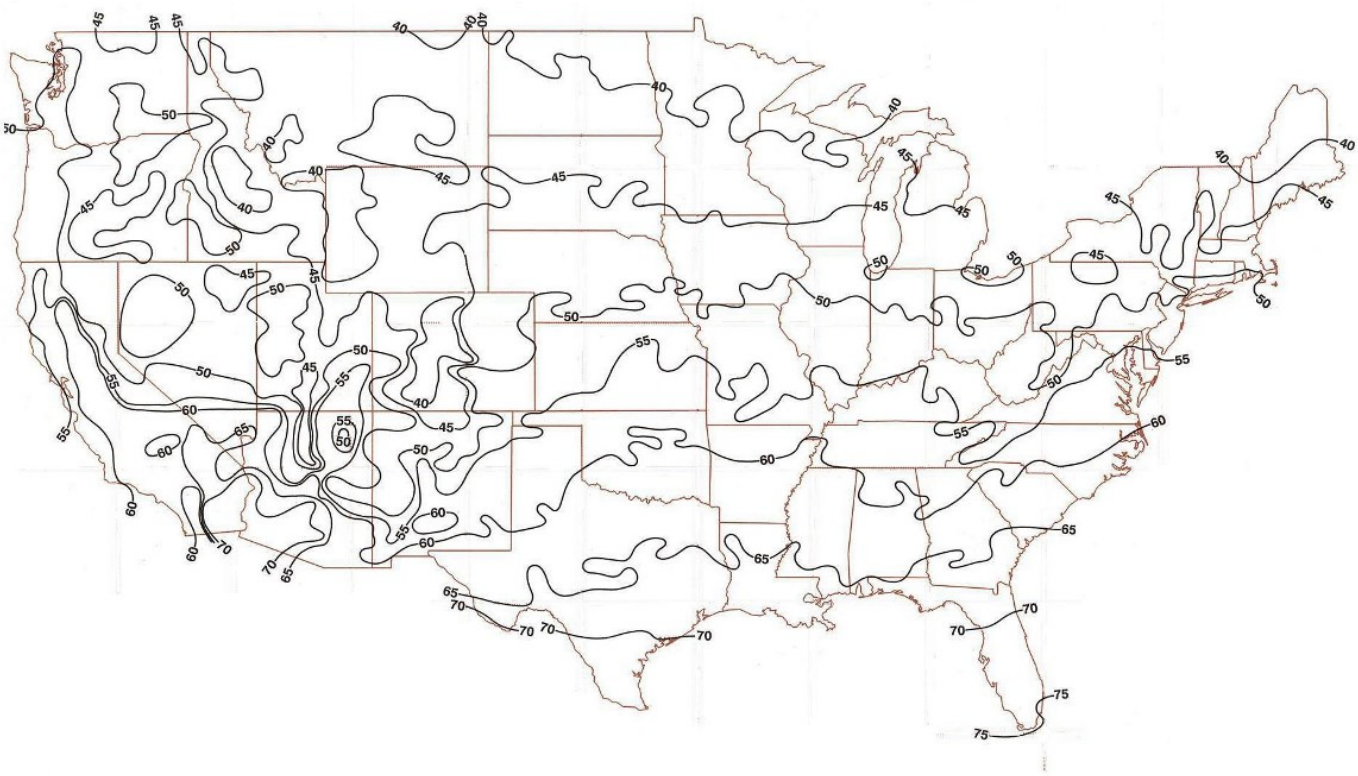

Figure 9. Mean Average Annual Temperatures for the Contiguous United States.

Below-Grade Insulation for Post-Frame Buildings

Minimizing Heat Transfer

David R. Bohnhoff, P.E.

Below-grade insulation for post-frame buildings is installed to prevent structural damage from frost heave and to minimize building operating costs associated with excessive heat transfer. Causes of frost heave and options for controlling it were presented in Part I of this article. Presented in this final portion of the article are requirements for heat transfer control, as well as design details and associated constructability issues for below-grade insulation of buildings with embedded posts.

Introduction

Heat is technically defined as flow of energy due to a temperature variance. In buildings that are warmed or cooled (known as *conditioned buildings*), control of this energy transfer is a major design element. Although we have a long history of using thermal insulation to control heat transfer through above-grade building assemblies, use of thermal insulation to control below-grade heat transfer is a more recent development.

As was noted in the first article, virtually all conditioned post-frame buildings feature a concrete slab. For this reason, the location of below-grade thermal insulation in post-frame buildings is generally described relative to the concrete slab.

Five common insulation placement options are shown in Figure 1. This includes systems that utilize exterior horizontal wing insulation (Figure 1a), systems that feature only vertical exterior insulation (Figure 1b), systems in which insulation is placed under the concrete slab (Figure 1c and Figure 1d), and systems with horizontal insulation only (Figure 1e).

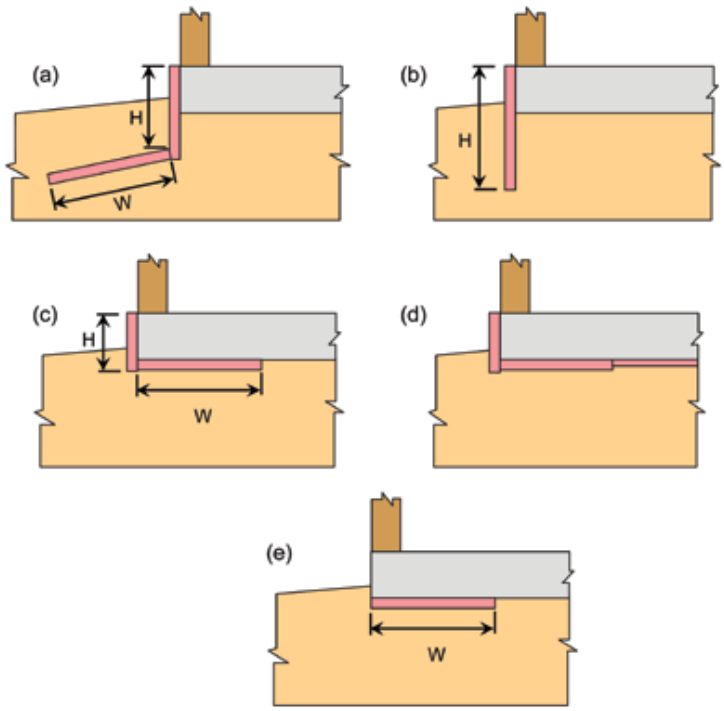

Figure 1. Below-Grade Insulation Options for a Concrete Slab-on-Grade.

(a) vertical and horizontal wing insulation, (b) vertical insulation only, (c) insulation on outside and underside of perimeter, (d) insulation on the outside edge and entire underside of slab, (e) horizontal insulation only

Quantifying Heat Loss

In order to efficiently insulate a building, an accurate estimate of heat transmitted through various elements of the building's thermal envelope (above-grade walls, windows, roofs, opaque doors, etc.) is required. The heat

Table 1. Assembly F-Factors for Slab-on-Grade Floors [a]

Type of Slab On Grade[b]	Insulation Description[c]	F-Factor, Btu/(h•ft•°F)[d]							
		R = 0	*R* = 5	*R* = 7.5	*R* = 10	*R* = 15	*R* = 20	*R* = 25	*R* = 30
Unheated	None	0.73							
	12 in. Horizontal w/o Thermal Break (TB)		0.72	0.71	0.71	0.71			
	24 in. Horizontal w/o TB		0.70	0.70	0.70	0.69			
	36 in. Horizontal w/o TB		0.68	0.67	0.66	0.66			
	48 in. Horizontal w/o TB		0.67	0.65	0.64	0.63			
	12 in. Vertical		0.61	0.60	0.58	0.57	0.57	0.57	0.56
	24 in. Vertical		0.58	0.56	0.54	0.52	0.51	0.51	0.50
	36 in. Vertical		0.56	0.53	0.51	0.48	0.47	0.46	0.46
	48 in. Vertical		0.54	0.51	0.48	0.45	0.43	0.42	0.42
	Fully Insulated Slab		0.46	0.41	0.36	0.30	0.26	0.23	0.21
Heated	None	1.35							
	12 in. Horizontal w/o TB		1.31	1.31	1.30	1.30			
	24 in. Horizontal w/o TB		1.28	1.27	1.26	1.25			
	36 in. Horizontal w/o TB		1.24	1.21	1.20	1.18			
	48 in. Horizontal w/o TB		1.20	1.17	1.13	1.11			
	12 in. Vertical		1.06						
	24 in. Vertical		0.99						
	36 in. Vertical		0.95						
	48 in. Vertical		0.91						
	Fully Insulated Slab		0.74						

[a] From ANSI/ASHRAE 90.1 Table A6.3. F-factors calculated assuming a 6-inch thick concrete slab with the bottom of the slab located at grade and a soil thermal conductivity of 0.75 Btu/(h•ft•°F).

[b] A heated slab-on-grade floor is a slab with a heating source either within it or below it. An unheated slab-on-grade floor is a slab that does not meet the definition of a heated slab-on–grade floor.

[c] 'Horizontal insulation w/o tb' refers to horizontal insulation without a thermal break (no vertical insulation on the outer slab edge) as shown in Figure 1e. Use vertical insulation values for cases shown in Figures 1a, 1b, and 1c, with vertical insulation length equal to the sum of lengths *H* and *W*. In all cases, the vertical portion of the insulation must extend to the top edge of the slab. For a slab to be fully insulated, the insulation must extend downward from the top of the slab and along the entire perimeter and completely cover the entire area under the slab.

[d] Interpolation of F-factors between R-values of insulation is allowed. Rated R-values are in units of h•ft^2•°F/Btu.

transmittance per unit time through such an element (including the transmission of heat through the boundary air films on both sides of the element) that is induced by a unit temperature difference between the environments on both sides of the element, is defined as the U-factor or thermal transmittance of the element.

Common units on the U-factor are Btu/(h•ft^2•°F). It follows that heat being transmitted through an element is obtained by multiplying the element's U-factor by the element's surface area and by the difference between inside and outside building air temperatures.

When air infiltration is blocked, heat transmission through an above-grade building element is fairly constant across the surface of the element; hence the U-factor does not vary appreciably with location. This is not the case for below-grade building elements. The amount of heat transmitted through a below-grade element depends on surrounding soil temperatures, which in turn depends on distances between the point in question and (1) the soil surface, (2) the ground water table, (3) surrounding buildings, and (4) other elements of the same building.

F-factors

Accurate determination of variations in heat transmission from location-to-location on a below-grade building element requires a three-dimensional thermal analysis with a fairly sophisticated piece of software. To avoid conducting such complex analyzes for every slab-on-grade floor, F-factors have been developed.

Table 2. Assembly F-Factors for Slab-On-Grade[a]

Soil Thermal Conductivity, Btu/(h•ft•°F)	Insulation R-value, h•ft^2•°F/Btu	F-Factors in Btu/(h•ft•°F)			
		Unheated Slab[b]			Heated Slab[b]
		2 feet of Vertical Insulation[c]	4 feet of Vertical Insulation[c]	Fully Insulated[d]	Fully Insulated[d]
0.50	5	0.44	0.42	0.38	0.60
	10	0.40	0.36	0.31	0.47
	15	0.38	0.33	0.26	0.39
0.75	5	0.58	0.54	0.46	0.74
	10	0.54	0.48	0.36	0.55
	15	0.52	0.45	0.30	0.44
1.00	5	0.71	0.66	0.54	0.87
	10	0.67	0.60	0.41	0.63
	15	0.65	0.57	0.33	0.49
1.50	5	0.94	0.85	0.64	1.11
	10	0.88	0.78	0.47	0.71
	15	0.86	0.75	0.37	0.55

[a] F-factors calculated assuming a slab floor placed directly on the earth with the bottom of the slab at grade line.
[b] A heated slab-on-grade floor is a slab with a heating source either within or below it. An unheated slab-on-grade floor is a slab that does not meet the definition of a heated slab-on-grade floor.
[c] Vertical insulation length is the sum of length H and W as shown in Figures 1a, 1b, and 1c. The vertical portion of the insulation must extend to the top edge of the slab.
[d] For a slab to be fully insulated, the insulation must extend downward from the top of the slab and along the entire perimeter and completely cover the entire area under the slab.

The F-factor for a slab is an approximation of the total amount of heat transmitted through the slab expressed per unit length of slab perimeter. Common units on the F-factor are Btu/(h•ft•°F). To estimate the heat being transmitted from an entire concrete slab-on-grade floor, the F-factor for the slab is simply multiplied by the slab's perimeter length and by the difference between inside and outside building air temperatures.

F-factors for concrete slabs-on-grade are determined via computer analyzes and depend on such design variables as soil thermal conductivity, soil thermal mass, insulation thermal resistance, insulation location, whether the slab is heated or unheated, slab position with respect to grade, location of the ground water table, slab area, slab shape, shading of the soil surface, and proximity to other building foundations that heat/cool the soil.

Table 1 contains F-factors for slab assemblies as published in Table A6.3 of ANSI/ASHRAE 90.1 *Energy Standard for Buildings Except Low-Rise Residential Buildings*. These F-factors are from research conducted by Ecotope Inc. of Seattle, WA. The slab model was developed in 1988 by Mike Kennedy, who, along with fellow Ecotope coworkers David Baylon and Jonathan Heller, conducted numerous simulations over the next couple of years. The Ecotope group established their F-factors using climate data for typical 8-month heating seasons in Washington state. Table 1 values are for a soil thermal conductivity of 0.75 Btu/(h•ft•°F). Table 2 contains F-factors generated by the Ecotope researchers for other soil thermal conductivities.

Nowhere in Tables 1 and 2 is an adjustment for slab shape or size mentioned. Consequently, a reader is inclined to believe that slab size and shape have only a minor influence on the magnitude of an F-factor. In reality, both slab size and shape can significantly impact the F-factor, as several researchers have shown. The F-factors in Tables 1 and 2 were obtained from simulations involving a residential-sized 30- by 45-foot slab, this since they were generated by Ecotope for use in estimating heat loss from residential buildings. Ironically, the Table 1 values are published in a document titled *Energy Standard for Buildings Except Low-Rise Residential Buildings*. During a recent conversion that I had with David Baylon on this topic, he warned me that the F-factors in Tables 1 and 2 would do an extremely poor job of predicting heat lost to the soil from very large slab-on- grade buildings (e.g., big box stores).

Insulation Location

Table 3. Thermal Conductivity of Unfrozen Soils

Dry Bulk Density, γ_d, lbm/ft^3	Porosity, η	Thermal Conductivity of Dry Soil, k_{dry}, Btu/(h•ft•°F)	Moisture Content, w, % Dry Basis	Saturation Ratio, S_r	Kersten Number, K_e	Thermal Conductivity of Moist Soil, k, Btu/(h•ft•°F)		
Fine-Textured Soils with Sand Fraction < 0.40						q = 0	q = 0.15	q = 0.30
80	0.527	0.125	0	0.000	0.000	0.12	0.12	0.12
			5	0.122	0.106	0.19	0.19	0.19
			10	0.243	0.391	0.36	0.38	0.37
			15	0.365	0.597	0.49	0.52	0.49
			20	0.487	0.734	0.57	0.61	0.58
			25	0.608	0.829	0.63	0.67	0.64
			30	0.730	0.899	0.67	0.71	0.68
			35	0.852	0.951	0.81	0.75	0.71
			40	0.973	0.992	0.90	0.77	0.74
100	0.408	0.163	0	0.000	0.000	0.16	0.16	0.16
			5	0.196	0.287	0.37	0.39	0.38
			10	0.392	0.632	0.62	0.67	0.63
			15	0.588	0.816	0.75	0.81	0.77
			20	0.784	0.924	0.83	0.90	0.85
			25	0.980	0.994	0.88	0.95	0.90
120	0.290	0.201	0	0.000	0.000	0.20	0.20	0.20
			5	0.331	0.548	0.68	0.74	0.69
			10	0.662	0.863	0.95	1.05	0.97
			15	0.993	0.998	1.06	1.18	1.10
Coarse Grained Soils With Sand Fraction > 0.40						q = 0.5	q = 0.7	q = 0.9
100	0.408	0.163	0	0.000	0.000	0.16	0.16	0.16
			5	0.196	0.452	0.62	0.68	0.75
			10	0.392	0.672	0.84	0.93	1.04
			15	0.588	0.812	0.98	1.09	1.22
			20	0.784	0.914	1.08	1.21	1.35
			25	0.980	0.993	1.16	1.30	1.45
120	0.290	0.201	0	0.000	0.000	0.20	0.20	0.20
			5	0.331	0.616	1.00	1.13	1.28
			10	0.662	0.854	1.31	1.49	1.70
			15	0.993	0.998	1.49	1.71	1.95
140	0.172	0.240	0	0.000	0.000	0.24	0.24	0.24
			5	0.652	0.848	1.66	1.94	2.26
			7.5	0.978	0.992	1.90	2.22	2.60

k = $(k_{sat} - k_{dry})\bullet K_e + k_{dry}$
K_e = $\exp\{\alpha\bullet[1-S_r^{(\alpha-1.33)}]\}$
k_{sat} = = thermal conductivity of saturated soil = $k_w^{\eta}\bullet k_s^{(1-\eta)}$
k_{dry} = thermal conductivity of dry soil = $-0.323\bullet\eta+0.295$ (Btu/(h•ft•°F))
α = 0.96 for course grained soils with sand fraction > 0.40
α = 0.27 for fine textured soils with sand fraction < 0.40
S_r = $(w/100)/[(\gamma_w/\gamma_d - (1/G_s)]$
w = moisture content, % dry basis
η = porosity = $1-\gamma_d/(G_s\bullet\gamma_w)$
γ_d = soil dry bulk density (dry unit weight)
γ_w = 62.4 lbm/ft^3 = water density

G_s = 2.71 = specific gravity of soil solids
k_w = thermal conductivity of water
= 0.34 Btu/(h•ft•°F)
= 1.28 Btu/(h•ft•°F) for ice
k_s = $k_q^{q}\bullet k_o^{(1-q)}$
= thermal conductivity of soil solids
q = quartz content, fraction of total solids
k_q = thermal conductivity or quartz
k_o = 1.2 Btu/(h•ft•°F) for q > 0.2
k_o = 1.7 Btu/(h•ft•°F) for q ≤ 0.2

Table 4. ASTM C578 Standard Specification for Rigid, Cellular Polystyrene Thermal Insulation

Insulation Type		Minimum Insulation Density, lbm/ft^3	Minimum R-value per Inch of Thickness, h•ft^2•°F/Btu	Minimum Compressive Strength at Yield or 10% Deformation[a], lbf/in^2	Allowable Bearing Capacity[b], lbf/ft^2	Maximum Water Vapor Permeance for 1-inch Thickness, U.S. Perms	Maximum Water Absorption by Total Immersion, Volume %
Expanded Polystyrene (MEPS)	II	1.35	4.0	15	N/A	3.5	3.0
	IX	1.9	4.2	25	1200	2.5	2.0
Extruded Polystyrene (XEPS)	X	1.35	5.0	15	N/A	1.5	0.3
	IV	1.6	5.0	25	1200	1.5	0.3
	VI	1.8	5.0	40	1920	1.1	0.3
	VII	2.2	5.0	60	2880	1.1	0.3
	V	3.0	5.0	100	4800	1.1	0.3

[a] The primary means for identifying ASTM C578 insulation type at a retail level is via the compressive strength at yield or 10% deformation. For example, Owens Corning's FoamulaR 150, 250, 400, 600 and 1000 are XEPS insulations with minimum compressive strengths of 15.0, 25.0, 40.0, 60.0 and 100.0 lbf/in^2, respectively and they are classified as ASTM C578 types X, IV, VI, VII and V, respectively.

[b] Allowable bearing capacity is based on ASTM C578 compressive strength at 10% deformation divided by a safety factor of 3.0 for conditions without cyclic loading (e.g., highway vehicle loading).

Not covering the outside vertical edge of a slab with insulation (Figure 1e) significantly increases heat transmission; for this reason, this case is treated independently in Table 1 from those with vertical insulation covering the outside edge of the slab (Figures 1a through 1d).

Vertical insulation length, as specified in the second column of Table 1 and in the header of Table 2, is equal to H as shown in Figure 1b or the sum of lengths H and W as shown in Figures 1a and 1c. In other words, Tables 1 and 2 draw no distinction between the insulation placements shown in Figures 1a-1c. As long as the insulations used in two different systems cover the vertical edge of the slab and have the same R-value and same total length (again, total length equals *H*+*W* in Figures 1a and 1c) the total heat transferred by the different systems is assumed to be identical. In reality, there are slight differences in heat transmission between the Figure 1a, 1b, and 1c locations, but the differences are not nearly as significant as the difference between one of these cases and a slab without the outside edge covered (Figure 1e).

Where a concrete slab directly overlies frost susceptible soils in a cold climate region, the insulation systems in Figures 1a and 1b are recommended.

Heated and Unheated Slabs

F-factors in Tables 1 and 2 are compiled for both heated and unheated slab-on-grade floors. A heated slab is one with a heating source either within or below it. Note that regardless of exactly where the heating source is located, the insulation is always assumed to be between the heating source and soil. An unheated slab-on-grade floor is a slab that does not meet the definition of a heated slab-on-grade floor.

The reason F-factors are higher for heated slabs is because a heated slab will be warmer than the design indoor temperature, and thus for a given indoor temperature, expect more overall heat loss from a heated slab.

Soil Thermal Conductivity

It is clear from Table 2 that soil thermal conductivity has a significant impact on F-factor values and thus care should be taken into account when estimating this value. Thermal conductivity *k* is the ability of a material to conduct heat, and although English units on thermal conductivity are identical to those of the F-factor, they really are not identical properties.

Table 3 contains *k* values for unfrozen soil that were obtained using the relationships at the bottom of Table 3. These equations were published by Sen Lu and colleagues (Lu 2007). To determine *k* with these equations only requires knowledge of the soil's dry bulk density γ_d, moisture content *w*, and quartz

content q. Soil dry bulk density and moisture content are used to calculate soil porosity η and degree of soil saturation S_r. Quartz content is used to calculate thermal conductivity of the soil solids k_s. Quartz content is the fraction of dry soil that is quartz, and can be approximated as the fraction of the dry soil mass that is sand (although this tends to overestimate k_s). Knowledge of the quartz content is important because quartz has a thermal conductivity measurably higher than other common soil minerals.

Soil thermal conductivity k is calculated using the thermal conductivity of dry and saturated soil (k_{dry} and k_{sat}, respectively) and a normalized thermal conductivity K_e (i.e., the Kersten number). Note that k_{dry} is calculated from soil porosity η, whereas k_{sat} is a direct function of the thermal conductivity of liquid water k_w and the thermal conductivity of soil solids k_s. The relative contributions that k_w and k_s make to k_{sat} is a function of soil porosity. The Kersten Number K_e is a function of the degree of soil saturation S_r and a soil texture dependent parameter α which is set equal to 0.27 for fine textured soils with a sand fraction less that 0.40, and to 0.96 for coarse grained soils with a sand fraction greater than 0.40.

It is evident from Table 3 values that soil thermal conductivity increases with increases in dry bulk density, soil moisture and quartz content. On average, k is greater for coarse soils, largely because of its quartz content and higher dry bulk densities.

At 32 degrees Fahrenheit ice has a thermal conductivity of 1.28 Btu/(h•ft•°F), which is almost four times that of liquid water at the same temperature. For this reason, frozen soil will have a higher thermal conductivity than unfrozen soil of the same moisture content. The difference is relatively minor at low moisture content levels, but can be significant for soils with low dry bulk densities and high moisture contents. To approximate the thermal conductivity of a frozen soil, use the same equations with k_w set equal to ice's thermal conductivity of 1.28 Btu/(h•ft•°F).

In the absence of soil property data, thermal conductivity is generally approximated as 0.75 Btu/(h•ft•°F).

Table 5. Prescriptive Perimeter Insulation Requirements for Slab-On-Grade Floors (ANSI/ASHRAE Standard 90.1-2007)

Slab-On-Grade Type[a]	Climate Zone[b]	Assembly Maximum F-Factor, Btu/(h•ft•°F)		
		Heated and/or Cooled Non-Residential Spaces[c]	Heated and/or Cooled Residential Spaces[c]	Semiheated Spaces[d]
Unheated	1, 2, 3	0.730	0.730	0.730
	4, 5	0.730	0.540	0.730
	6	0.540	0.520	0.730
	7	0.520	0.520	0.730
	8	0.520	0.510	0.730
Heated	1, 2	1.020	1.020	1.020
	3	0.900	0.900	1.020
	4, 5	0.860	0.860	1.020
	6	0.860	0.668	1.020
	7	0.843	0.668	0.900
	8	0.688	0.668	0.900

[a] A heated slab-on-grade floor is a slab with a heating source either within or below it. An un-heated slab-on-grade floor is a slab that does not meet the definition of a heated slab-on-grade floor.
[b] See Table 6 and Figure 2.
[c] A heated space within a building is a space heated by a system whose output capacity in Btu/h per square foot of floor area is greater than or equal to: 5 for climate zones 1 and 2; 10 for climate zone 3; 15 for climate zones 4 and 5; 20 for climate zones 6 and 7; and 25 for climate zone 8. A cooled space within a building is a space cooled by a system with a sensible output capacity exceeding 5 Btu/h per square foot of floor space.
[d] A semiheated space within a building is a space heated by a system whose output capacity is greater than or equal to 3.4 Btu/h per square foot of floor area but less than required to be a heated space as given in Footnote c.

Insulation types

Control of below-grade heat transfer was revolutionized by the development of rigid, plastic foam board insulations. Base plastics include polystyrene, polyisocyanurate (also known as polyiso), and polyurethane. There are two main categories of polystyrene products: extruded/expanded polystyrene (XEPS or XPS) and molded/expanded polystyrene (MEPS or EPS). These two products are discussed in more detail here because they are the most widely used products for below-grade insulation. Polyisocyanurate and polyurethane boards are similar in formulation. They tend to be more expensive than polystryrene-based boards, but can withstand higher temperatures than polystyrene, thus making them more attractive for roofing applications.

XEPS is manufactured by melting polystyrene beads and mixing the resulting liquid with special additives

Table 6. Climate Zone Delineation Criteria

Climate Zone[a]	Climate Zone Description	Annual degree-days[b], °F•day
1	Very Hot	Greater than 9000 CDD50
2	Hot	6300 - 9000 CDD50
3	Warm	Less than 6300 CDD50 and Less than 3600 HDD65
4	Mixed	Less than 4500 CDD50 and Less than 5400 HDD65
5	Cool	5400 - 7200 HDD65
6	Cold	7200 - 9000 HDD65
7	Very Cold	9000 - 12600 HDD65
8	Subarctic	Greater than 12600 HDD65

[a] See Figure 2.
[b] CCD50 = annual cooling degree days calculated using a 50F base temperature. One cooling degree day (CDD) accumulates for every degree the average daily (24 hr) temperature is below 50F. HDD65 = annual heating degree days calculated using a 65F base temperature. One heating degree day (HDD) accumulates for every degree the average daily temperature is below 65F. For a 5 day period with average outside daily temperature of 64, 55, 46, 57, and 62F, a total of 38 CCD50 accumulates (38= 14 + 5 + 0 + 7 + 12)

and a blowing agent inside an extruder under a very specific temperature and pressure. The resulting hot viscous liquid is extruded through a die into a reduced-pressure environment, upon which the gaseous blowing agent expands, resulting in the formation of a uniform closed-cell foam sheet with smooth skins. Common trade-named products include Dow Chemical's Styrofoam (blue), Owen Corning's Foamular (pink), Pactiv Corporation's GreenGuard (green) and DiversiFoam Products' CertiFoam (yellow). MEPS board is produced from polystyrene resin beads that contain microscopic cells filled with a blowing agent (usually pentane or butane). The beads are exposed to steam under controlled pressure. The heat from this steam simultaneously softens the cell walls and expands the blowing agent, causing individual beads to increase in volume by up to forty times. After a brief holding period to allow stabilization, the expanded closed-cell beads are poured into a large block mold. Steam is injected into the mold, and heat and pressure further expand and fuse the beads into a molded block. These blocks are cooled and then cut with hot wires to make the thinner sheets used for insulation. Because of its manufacturing process, MEPS is frequently referred to as bead board. Most people are familiar with EPS as a packaging product for which it is ideal, because it is lightweight (reduced shipping cost), shock absorbent, non-abrasive, not weakened or damaged by moisture exposure, and insulates package contents from temperature changes.

Specifications for various types of MEPS and XEPS are compiled in Table 4. Note that for a given density, XEPS exhibits a higher thermal resistance, higher compressive strength, lower water vapor permeance, and lower water absorption than MEPS. The lower properties for MEPS are attributable to microscopic pores between the closed-cell expanded beads. Because of the higher water permeance and adsorption of MEPS, there was a widespread belief among many in the building industry that MEPS deteriorated in the presence of wet soils subjected to freeze-thaw cycles. These concerns subsided after some U.S and Canadian field studies conducted in the 1990s showed long term performance of MEPS to be just as good as XEPS. Today, many of the insulated concrete

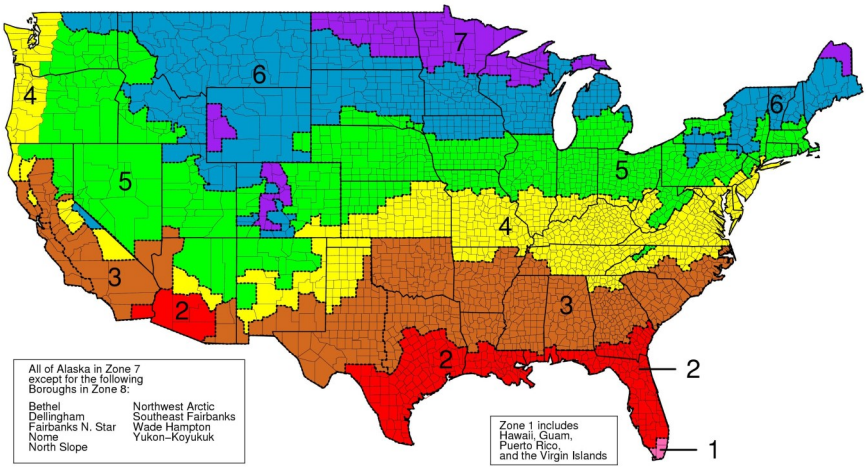

Figure 2. Climate Zones for the Contiguous United States

forms (ICFs) used for foundations are MEPS products. In addition to the physical and thermal properties compiled in Table 4, it should be noted that MEPS and XEPS are unaffected by common soil acids, will not support mildew and fungus growth, and are decay and corrosion resistant.

The downsides of MEPS and XEPS are that they are deteriorated by ultraviolet radiation, easily damaged by certain solvents, melt at temperatures above 250 degrees Fahrenheit, incompatible with certain thermoplastics (polystyrene insulations are known to draw plasticizers out of thermoplastic membranes, causing permanent degradation), flammable, and subject to termite infestations. Where termite infestations are a potential problem, external applications of MEPS and XEPS may be prohibited, the insulations may need to be encased in concrete, or an inspection gap may be required between the insulation and wood-based building materials.

To combat physical damage and deterioration via sunlight, exposed exterior MEPS and XEPS are generally covered with a stucco coating, pressure-treated wood, brick, or aluminum flashing. When installed on the interior of a building, the flammability of these insulations requires that they be protected by a suitable "thermal barrier" or "ignition barrier" in accordance with the applicable code. In most cases, this need is met with a half-inch thick gypsum wall board covering.

Code Requirements

Thermal insulation requirements embodied in energy conservation-related codes are largely based on the ANSI/ASHRAE Standard 90.1-2007. There are two different "compliance paths" outlined in ASHRAE 90.1 -07: a prescriptive building envelope option and a building envelope tradeoff option.

Designers following the prescriptive building envelope option must make sure that each element of the building thermal envelope has a thermal transmittance (i.e., U-factor, F-factor) less than its code-prescribed value. Prescriptive thermal insulation requirements for slab-on-grade floors from ASHRAE 90.1-07 Table 5.5 are given in Table 5.

Designers following the ASHRAE 90.1-07 building envelope trade-off option must make sure the total heat transmission through the entire building thermal envelope does not exceed a code specified value. Via this option, a designer can utilize a concrete slab-on-grade with an F-factor that exceeds the applicable maximum prescribed value listed in Table 5 as long as the total performance of the thermal envelope meets code requirements. The trade-off option is popular as it provides greater flexibility in design, which makes it easier to lower overall building cost. Implementation of the building envelope trade-off option is facilitated with spreadsheet programs or special computer programs (e.g., REScheck, COMcheck).

Climate Zones

As indicated in Table 5, thermal insulation requirements depend on temperature differences across a building envelope. The greater and more prolonged these temperature differences, the greater the insulation needed, and the lower the prescribed maximum F-factor. This explains why the prescribed values in Table 5 are lower for heated and cooled spaces than for semi-heated spaces, and why prescribed F-factors are lower for northern climate zones.

Climate zones referred to in Table 5 are shown in Figure 2 and defined in Table 6. Current zone differentiation is based on work done by Robert Briggs, Robert Lucas, and Todd Taylor of the Pacific Northwest National Laboratory (PNNL). The 2004 *International Energy Conservation Code (IECC) Supplement* was the first major publication to adopt the PNNL climate zones and it was subsequently adopted by ANSI/ ASHRAE 90.1.

As Table 6 indicates, climate zones are primarily based on heating and cooling degree days. Although degree days dictate above grade insulation requirements, insulation requirements for controlling below-grade heat transfer are largely dictated by soil temperatures and thermal conductivities. Soil temperatures, in turn, are directly related to annual average dry bulb temperatures that are graphically displayed in Figure 3. A comparison of the climate zones in Figure 2 with the isotherms in Figure 3 explains why climate zones can be used to approximate soil temperatures.

Example Selection

In accordance with ASHRAE 90.1-07 prescriptive building envelope requirements, what insulation options (total vertical plus horizontal insulation length and insulation R-value) can be used for an unheated concrete slab that supports a heated/cooled building located in climate zone 7 where the soil has a thermal conductivity of 0.75 Btu/(h•ft•°F)?

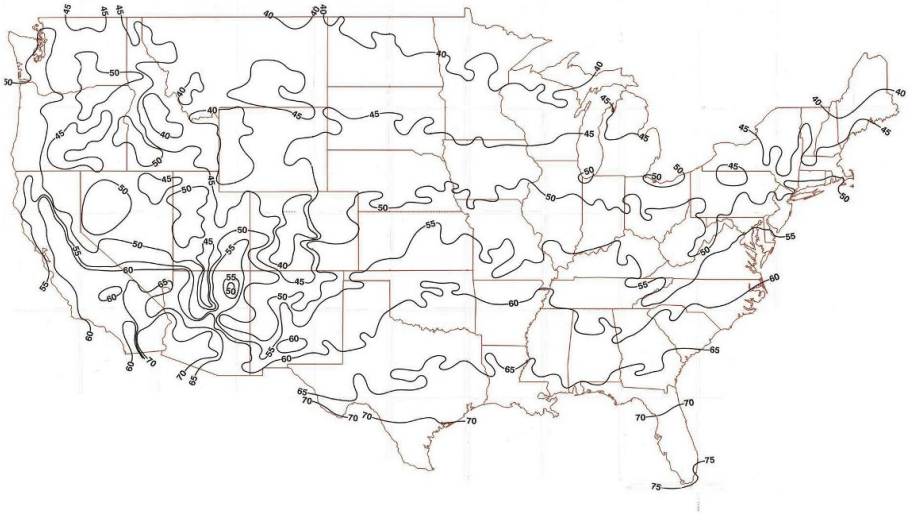

Figure 3. Mean Average Annual Temperatures for the Contiguous United States.

Table 5 requires that the F-factor not exceed 0.520 Btu/(h•ft•°F). Based on Table 1, this requirement can be met with 24 inches of R15 insulation (F-factor = 0.52), 36 inches of R10 insulation (F-factor = 0.51), or 48 inches of R7.5 insulation (F-factor = 0.51). Alternatively, the entire slab can be insulated with R5 insulation (F-factor = 0.46). Note that the 0.520 Btu/(h•ft•°F) requirement cannot be met with an application featuring only horizontal insulation (Figure 1e).

Insulating Heated Slabs

The rule of thumb for insulation of heated slab-on-grade floors is to increase by 5 h•ft^2•°F/Btu the R-value that would be prescribed for the slab if it was not heated, and also install insulation with a minimum R-value of 5 h•ft^2•°F/Btu under the remainder of the slab. Please note that a number of energy specialists recommend that the remainder of a heated slab be insulated with an R-value of no less than 10 h•ft^2•°F/Btu

Installing Below-Grade Insulation

Figures 4 to 6 contain construction details for below-grade insulation of buildings with embedded posts. Prior to discussing these details, I would point out that these details are just as applicable to posts mounted on concrete piers, with the exception that thermal breaks should be added between any concrete slab and concrete pier. I would also note that below-grade insulation details for buildings with posts that are not embedded or are not placed on piers are not included here. Such posts are either placed on concrete stemwalls or slab-on-grade foundation systems, and common practices as they relate to insulating such concrete foundations are widely published elsewhere.

Figure 4 illustrates an insulation system that would not be placed in contact with frost-susceptible soils since soil located directly under the perimeter of the concrete slab could freeze and heave. There is absolutely no problem with this system as long as the concrete slab is placed on well-drained, nonfrost-heave susceptible soils, and the post footing is placed below the anticipated frost depth. From both a cost and a constructability perspective, the system shown in Figure 4 may be the best way to insulate a building with embedded posts. With the right soils, it is the ideal system for buildings with heated concrete slabs. When installing a heated slab, I strongly recommend placing insulation with a minimum R-value of 10 h•ft^2•°F/Btu under the entire interior of the slab. In climate zones 3 and higher, energy codes will generally require that insulation under the exterior perimeter and along the outside edges of a heated slab have an R-value greater than 10 h•ft^2•°F/Btu.

Figures 5 and 6 both show systems that can be used for conditioned post-frame buildings with embedded posts and slabs overlying frost susceptible soils. In Figure 5, girts are outset and insulation near grade

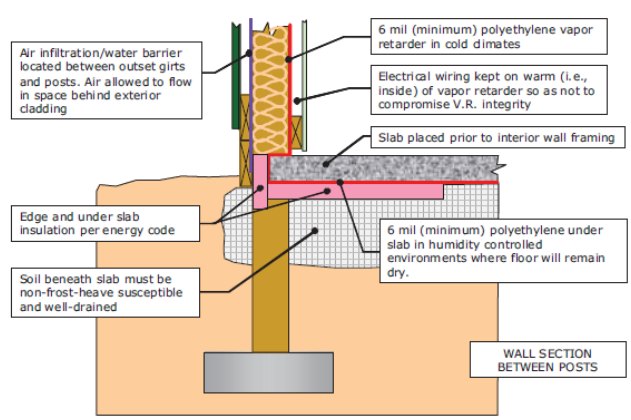

Figure 4. Insulation System for a Heated Post-Frame Building with Embedded Posts, Outset Girts, and Slab Edge Insulation Protected by Splash Plank.
This system requires slab placement on well-drained, non-frost-heave susceptible soil.

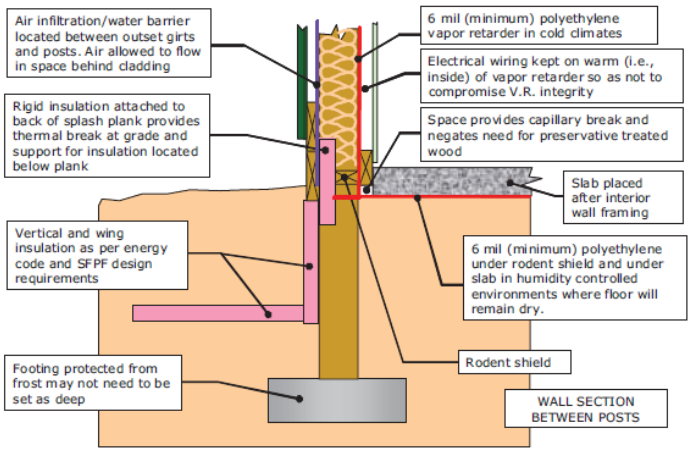

Figure 5. Insulation System for a Heated Post-Frame Building With Embedded Posts, Outset Girts, and Insulation Near Grade Protected by Splash Plank

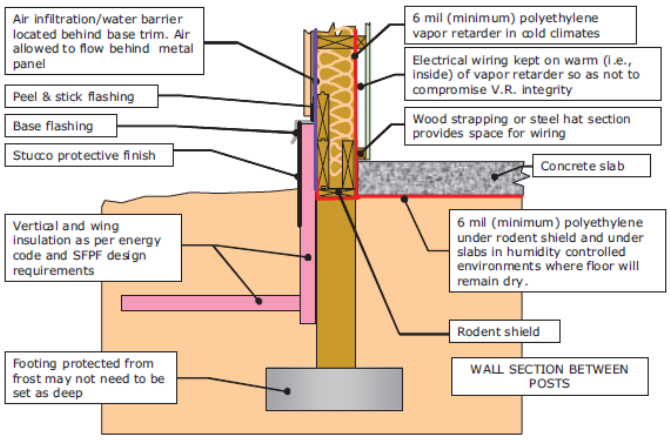

Figure 6. Insulation System for a Heated Post-Frame Building With Embedded Posts, Inset Girts, and Insulation Located Outside Base Girts

is protected by splash plank. In Figure 6, girts are inset and insulation is located outside base girts.

The problem with using the insulation systems in Figures 5 and 6 for buildings with embedded posts is two-fold. First, they negate one of the major advantages of the embedded post foundation system and that is minimal excavation. Since below-grade vertical and horizontal wing insulation cannot be installed until after posts (or piers) have been embedded, when is the excavation for the insulation made?

One option is to begin building construction by digging a trench at a depth and width that will accommodate the horizontal wing insulation as well as compaction of soil on the inside of the vertical insulation. Once the trench is dug, post holes can be augered (this has the benefit of decreasing post-hole augering depth).

A second option is to trench alongside the exterior of the building after the exterior shell has been completed, but before interior concrete work has begun. The problem with this option is that there's no inside trench in which to run the typical compactor that would be used to compress soil against the inside of the vertical insulation.

A third option is to construct the building with the grade at least a full foot lower than the finish grade, with insulation placement done just before soil is added to bring it up to the final elevation. While this requires fill material to be placed inside the structure after the shell is complete, such a sequence may actually be beneficial in buildings featuring extensive under-slab plumbing, electrical, HVAC and other systems.

The second problem with the insulation systems in Figures 5 and 6 when used in buildings with embedded posts involves installation of the vertical insulation. More specifically, how is the soil adequately compacted on both sides of the vertical insulation without compromising the integrity of the insulation system?

With concrete stemwall and slab-on-grade foundations, the vertical insulation is simply compacted against a vertical concrete surface. Such a surface does not exist between embedded posts. When compacting soil, it is important to keep the vertical insulation straight so as to maintain a tight joint between it and the horizontal wing insulation. This can be accomplished by temporarily fastening an inset girt against the inside of the vertical insulation while compacting soil against the outside surface.

Regardless of the insulation method used, make sure that a durable, opaque, weather-resistant covering or coating is used to protect exposed insulation from ultraviolet radiation, physical damage, and other sources of deterioration. Also make sure that compressive loads on insulation materials supporting building foundation loads do not exceed the allowable bearing capacities given in Table 5, and that the surface upon which any horizontal insulation is laid has been struck level. Production of a compact and level surface often requires that the surface be worked with a straight edge in between passes with a mechanical compactor.

Vapor Retarding Membranes and Sand Layers

Vapor retarding membranes are placed below

concrete slabs to minimize the diffusion of water vapor and other gases from the soil into the building environment. When properly installed they can reduce indoor humidity levels; prolong the life of flooring materials; decrease mold growth potential; and reduce indoor concentrations of radon, methane and other unwanted gases.

In typical construction, a vapor retarding membrane is placed over a layer of pea gravel that serves as a capillary break between the soil and the membrane. After placing the membrane over the pea gravel, some builders cover it with a layer of sand and then place concrete on top of the sand layer. Depending on the intended purpose of the sand layer, it may be referred to as a blotter, cushion layer, or protection course.

The sand layer acts as a blotter when it is relied on to absorb excess water during concrete placement in an effort to reduce bleed water at the concrete surface. Although the term blotter is appropriate for this application, this is not a sound reason for installing a sand layer. Excess bleed water is a symptom of a water-to- cement ratio that is too high, and a sign that the concrete will have a relatively low strength and high permeability when cured.

Additionally, it has been shown to take slabs placed over sand 3 to 4 times longer to dry down to the same point as slabs placed directly on membranes. A prolonged wet slab can negatively impact the quality and schedule of flooring installations. In the end, a builder is much better off spending money on concrete additives and proper concrete curing than on a “blotter”.

Sand layers serve as cushion layers and protection courses when they are installed to prevent accidental puncture of membranes during steel reinforcing and concrete placement. In this case, use of a sand layer can be avoided by using a thicker or more puncture resistant membrane (e.g., 10 or 15 mil polyethylene meeting ASTM E1745 Class A membrane requirements), or in the case of insulated slabs, by placing rigid insulation on top of the membrane. Please note that by placing the membrane on the moist side of the insulation the insulation is less likely to be compromised by water absorption.

Finally, an unlikely but potential problem introduced by placing a sand layer between a vapor retarding membrane and a concrete slab is that should water ever find its way into the sand layer, the membrane will help ensure that the water spreads out in the sand layer and stays there until it diffuses up through the concrete. In this case, the membrane is contributing to a problem that it was intended to help solve.

Summary

Use of F-factors for calculating heat transmission from concrete slab-on-grades was presented. These F-factors are a function of many variables including soil thermal conductivity, insulation thermal resistance, insulation location, whether the slab is heated or unheated, slab position with respect to grade, and total slab area and shape. Methods for calculating soil thermal conductivity were overviewed along with physical and thermal properties of MEPS and XEPS insulations. ANSI/ASHRAE Standard 90.1-2007 insulation requirements for concrete slab-on-grades were introduced, and three details for below-grade insulation of post-frame buildings with embedded posts were discussed, along with dis-advantages of introducing a sand layer between a concrete slab and an underlying vapor retarding membrane.

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Simplified Lateral Design of Post-Frame Buildings

Donald Bender, P.E. and Drew P. Mill

Post-frame construction is growing across a variety of building markets because of advantages related to cost, reliability and ease of construction. Much of the structural efficiency of post-frame buildings is attributed to diaphragm action distributing lateral loads (e.g., wind and seismic forces) to the shear walls of the buildings. When embedded posts are used in the foundation system, the frames and the roof diaphragm interact to resist lateral loads. Sophisticated design methods have evolved to account for the interaction between frames and diaphragm (e.g., ASAE 2003; Bohnhoff 1992a and 1992b; Anderson et al. 1989), and to design embedded post foundations with a range of detailing and soil behavior assumptions (e.g., ASAE 2005; McGuire 1998; Meador 1997). Designers that specialize in post-frame construction are well acquainted with these design tools. However, these design methods are not readily available or familiar to structural engineers with limited experience in post-frame; hence limiting the expansion of post-frame construction. As post-frame construction grows into new markets, rational, simplified design methodologies that can be quickly learned and economically implemented by design and building regulatory professionals are needed. The objective of this article is to present a simplified design method that provides conservative designs for roof diaphragms, shear walls, post members and embedded post foundations. Hence a structural engineer with a limited number of post-frame building projects per year can justify the cost of learning the design method.

Design Overview

A good technical resource for diaphragm and shear wall design for light-frame wood construction is APA Publication L350A, *Diaphragms and Shear Walls—Design/Construction Guide* (APA Engineered Wood Association 2007). Lateral design is the same for post-frame and light-frame wood construction, except for the following:

- If posts are embedded, the distribution of lateral loads between the foundation and the roof diaphragm is changed. In this article, we conservatively ignore the contribution of the frame because it is much less stiff than the diaphragm, and we obtain solutions that are conservative and easier to comprehend.
- Standardized diaphragm and shear wall design capacities are available for light-frame construction (e.g., ANSI/AF&PA SDPWS–2008 *Special Design Provisions for Wind and Seismic* (AWC 2008). Less data are available in the public domain for post-frame diaphragm and shear wall constructions that use metal cladding on wood framing. The National Frame Building Association (NFBA) is currently working to develop the standardized diaphragm and shear wall data needed by designers.

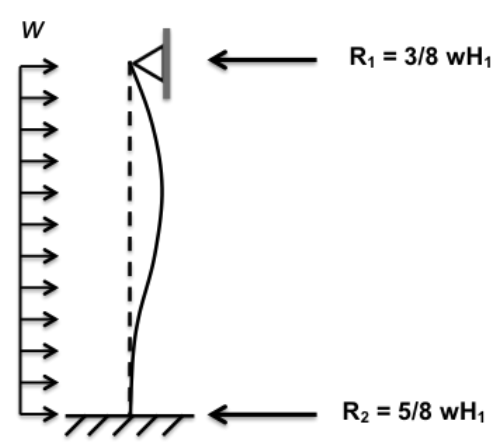

Figure 1. Propped Cantilever Analog for Rigid Roof Design

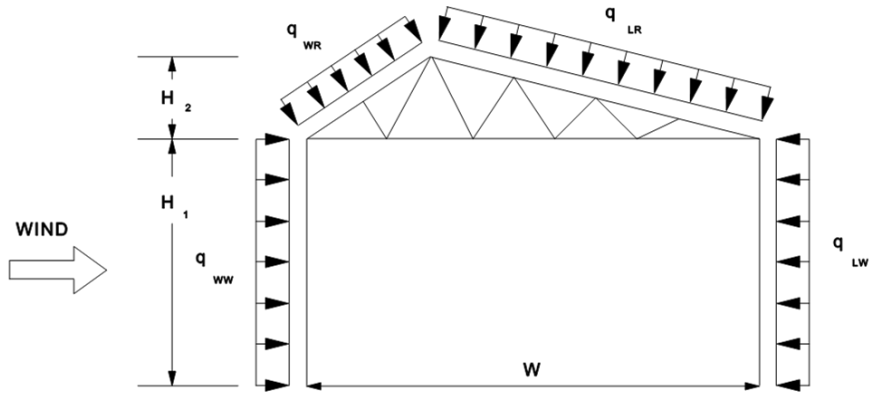

Figure 2. Wind Loading on Post-Frame Building

- Post-frame roof diaphragms have repetitively framed purlins that can share chord forces, which has a significant impact on chord member and splice connection design.

Determining Roof Diaphragm Forces Using the Rigid Roof Design Method

The rigid roof design method (Bender et al. 1991) is conservative with respect to roof diaphragm design because a propped cantilever analog is assumed, as shown in Figure 1. The pin supporting the top of the post represents an infinitely stiff roof diaphragm, thus attracting load to the diaphragm. More complicated analysis procedures are available that model the diaphragm as a spring supporting the top of the post, resulting in lower diaphragm loads.

Figure 2 shows a hypothetical wind loading on a post-frame building. The resulting unit shear for the building is given by:

$$v = \frac{K(q_{ww} - q_{LW})H_1 L + (q_{WR} - q_{LR})H_2 L}{2W}$$

where

v = unit roof shear intensity (lb/ft)
K = *3/8* for embedded posts, *1/2* for surface-mounted posts
q_{WW} = design windward wall pressure (+ sign for inward pressure, - sign for outward pressure) (psf)
q_{LW} = design leeward wall pressure (+ inward, - outward or suction) (psf)
q_{WR} = design windward roof pressure (+ inward, - outward or suction) (psf)
q_{LR} = design leeward roof pressure (+ inward, - outward or suction) (psf)
H_1 = side-wall height (ft)
H_2 = roof height (ft)
W = building width (ft)
L = building length (ft)

At this point, a diaphragm construction can be selected to meet the conservative estimate of unit shear demand. Some allowable design values are available for metal-clad wood-framed diaphragms and shear walls in the Post-Frame Building Design Manual (NFBA 1999), and NFBA is currently working to develop a standardized design database. Another option is to use wood panels on wood framing; design data can be found in the *Special Design Provisions for Wind and Seismic* AWC SDPWS–2008 standard (AWC 2012).

Shear Wall Design

If the shear wall has no openings, simply use the unit shear calculated for the roof diaphragm and select a shear wall construction to carry the load.

If the shear wall has an opening such as an overhead door, the segmented shear wall approach can be used where the end of each wall segment has a hold-down (or post). The unit shear demand is given in the following equation:

$$v_{shearwall} = \frac{V_{\max}}{(W_{bldg} - W_{opening})}$$

where

$v_{max} = v\,W$
W_{bldg} and $W_{opening}$ as illustrated in Figure 3.

At this point, a shear wall construction can be selected to meet the unit shear demand.

Roof Diaphragm Chord Forces

Usually the perimeter chords are assumed to resist all of the bending moment in light-frame wood diaphragms. In post-frame roof diaphragms, roof purlins can be assumed to share the chord forces as described by Pollock et al. (1996) and illustrated in Figure 4.

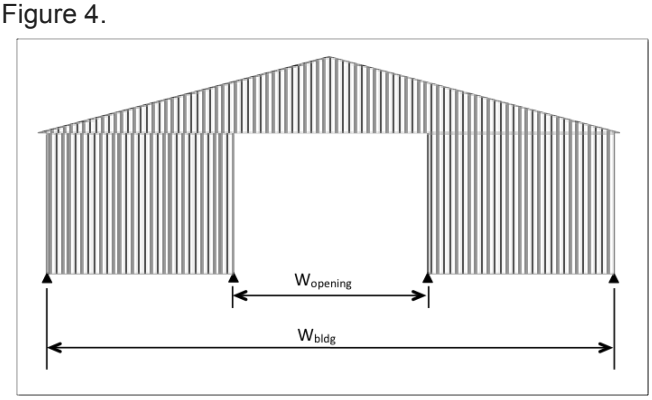

Figure 3. Shear Wall with Opening

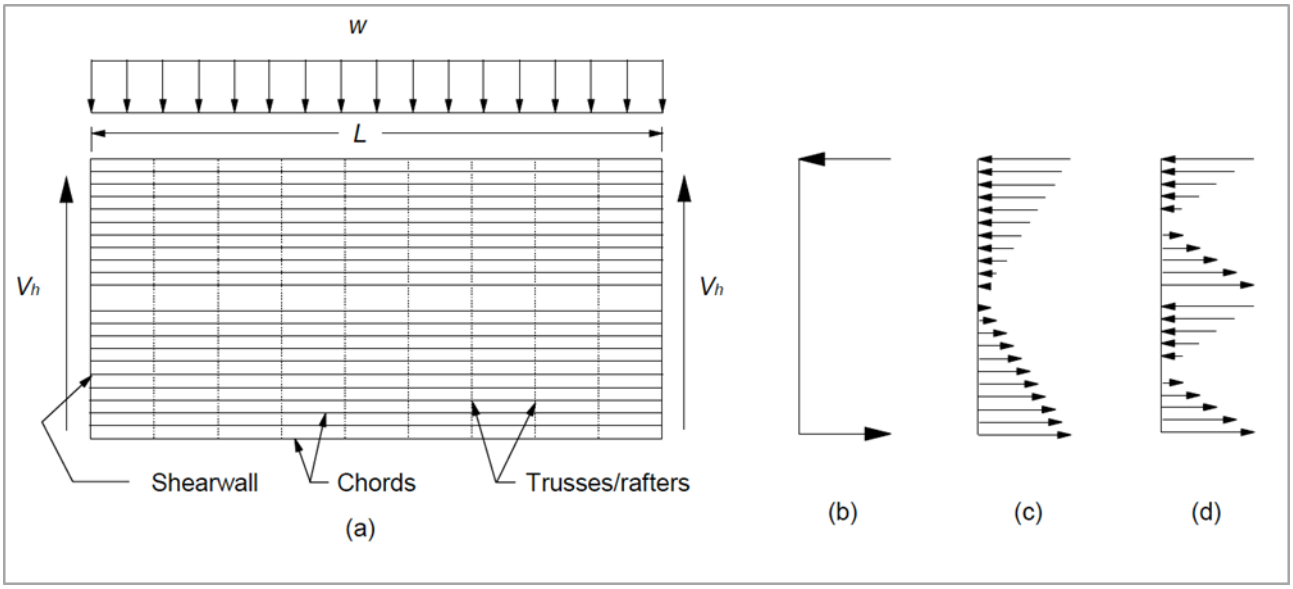

Figure 4. Diaphragm Under Uniform Load

(a) Plan view of a diaphragm under a uniform load, *w*. Chord force distribution when (b) moment is resisted by edge chords only, (c) chord force distribution is linear, and (d) chord force distribution is linear, but diaphragm halves are assumed to act independently in resisting moment (NFBA, 1999).

Procedure for calculating chord forces

1. Solve for maximum shear force, V_{max}, in the roof diaphragm using the rigid roof equation.

2. Calculate the resulting uniform load, *w*, on the roof diaphragm, assuming the load is evenly distributed along the building length.

$$w = 2V_{max}/L$$

3. Solve for the maximum bending moment, *M*, in the roof diaphragm. Engineering judgment is required with regard to the end conditions of the roof diaphragm. If a simple beam with pin and roller supports is assumed, the maximum moment is at the mid-length of the building as follows:

$$M = wL^2/8$$

If a beam with fixed conditions is assumed, the maximum moment will occur at the ends of the building as follows:

$$M = -wL^2/12$$

4. Solve for the maximum axial force on the perimeter purlin, T_n, using the following equation

$$T_n = \frac{Md}{\sum_{i=1}^{n}\left[d - 2(n-i)s(\cos\theta)\right]^2}$$

M = bending moment in roof diaphragm (ft-lb)
d = diaphragm depth (ft) (roof span)
i = purlin number, starting from the ridge and working to the eave
n = number of purlins from ridge to eave (one side of the roof and not counting the ridge purlin)
s = purlin spacing (ft)
θ = roof slope

The NFBA's Post-Frame Building Design Manual used the approach in Step 4 to create a convenient design table (see Table 1). In this case, the maximum chord force is given by:

$$T_n = M\,\alpha\,/d$$

Finally, the maximum chord force T_n is used to size the chord members and splices.

Post Member Forces and Embedded Foundation Design

Post Member Design

Maximum eave deflection will occur at the mid-length of a symmetric building, so this is usually the critical frame with respect to post member design and required embedment. Using the propped-cantilever model, we can calculate the maximum positive and negative (ground line) moments on the post, yet this simple structural analog does not allow any eave deflection.

Table 1. Reduction Factor, α, for Axial Force in Edge Chords

n^a	α	n^a	α	n^a	α	n^a	α
2	1.000	12	0.423	22	0.249	32	0.176
3	1.000	13	0.396	23	0.239	33	0.171
4	0.900	14	0.371	24	0.230	34	0.166
5	0.800	15	0.350	25	0.222	35	0.162
6	0.714	16	0.335	26	0.214	36	0.158
7	0.643	17	0.314	27	0.206	37	0.154
8	0.583	18	0.298	28	0.200	38	0.150
9	0.533	19	0.284	29	0.193	39	0.146
10	0.491	20	0.271	30	0.187	40	0.143
11	0.455	21	0.260	31	0.181	41	0.139

[a] *n* is the total number of purlins in the diaphragm.

In a real building, the eave will deflect an amount Δ_{eave} under lateral loading as shown in Figure 5a. The eave deflection will cause a negative moment in the post as shown in Figure 5b. By superposition, we can solve for the maximum moments in the positive and negative regions of the post.

Bending Moment for Propped Cantilever with Uniform Load

From beam tables, we can find the maximum positive and negative moments for a propped-cantilever member with uniform load as follows:

$M^+_{max} = 9wH_1^2/128$ (occurs at 3/8 L down from top of post)

$M^-_{max} = wH_1^2/8$ (occurs at ground line)

Bending Moment for Cantilever with Point Load

If we know the eave deflection Δ_{eave} at building mid-length,1 we can solve for the force *P* that would cause that deflection using the following equation:

$$\Delta_{eave} = PH_1^3/3EI$$

Solving the equation for *P*, and then substituting $M = PH_1$, we have the equation for bending moment caused by eave deflection Δ_{eave}

$M^-_{max} = PH_1 = 3\Delta_{eave}EI/H_1^2$ (at ground line)

$M^- = 3/8\ PH_1 = 9\Delta_{eave}EI/8H_1^2$ (at 3/8 H_1 down from top of post)

Combined Moments Using Superposition

The maximum net moment in the positive region occurs at approximately $3/8H_1$ from the top of the post. Note the difference in signs of the two moments.

$$M^+_{max} = 9wH_1^2/128 - 9\Delta_{eave}EI/8H_1^2$$

Similarly we solve for the maximum negative moment at the ground line:

$$M^-_{max} = wH_1^2/8 + 3\Delta EI/H_1^2$$

The ground line moment is needed to calculate post embedment depth and often controls member design.

Note that all relevant load combinations should be checked when designing any member. For post-frame posts, a combination of gravity and lateral loads typically controls. Gravity loads are straightforward to calculate.

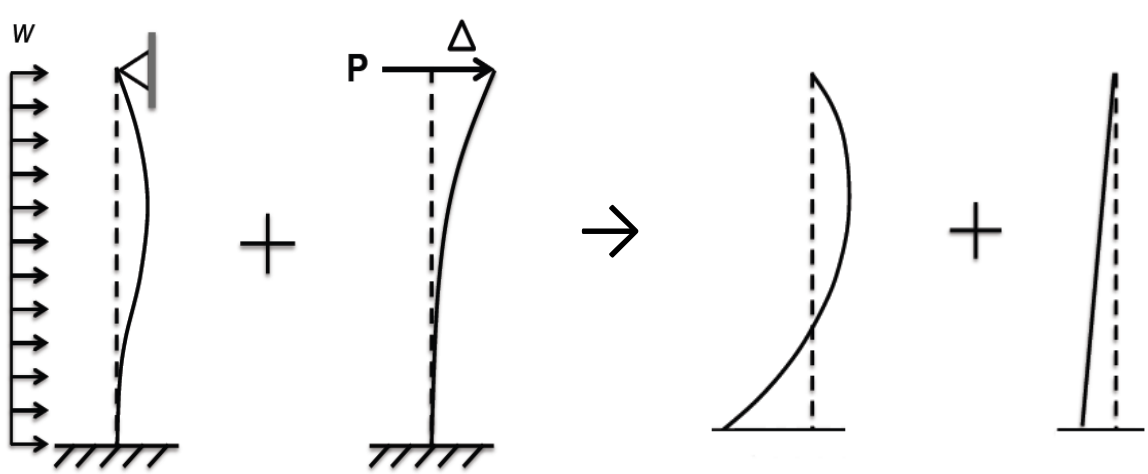

Figure 5. Superposition of (a) Structural Analogs, and (b) Resulting Moments

The post member design is accomplished using the combined bending and axial compression Equation 3.9-3 in the ANSI/AWC NDS-2012 (AWC 2012).

An alternate form to calculate post moments is presented on page 9-7 of the *Post-Frame Building Design Manual* (NFBA 1999). This approach gives equations to calculate shear and moments at different points in a post by summing forces based on statics. These equations yield identical results to that of the preceding equations.

Calculating Eave Deflection

Calculating eave deflection is required to determine the maximum post moment. Pope et al. (2012) present a three-term equation to predict diaphragm deflection that includes deflection contribution from bending of the diaphragm framing and chord slip which is presented as

$$\delta_{dia} = \frac{15vL^3}{4EAsn(n+1)} + \frac{0.25vL}{1000G_a} + \frac{\Delta_c}{W}\sum x_i$$

v = applied unit shear (lb/ft),
L = diaphragm length (ft),
E = modulus of elasticity of the diaphragm chords (psi),
A = cross-sectional area of the chords (in2),
s = chord spacing (ft),
n = number of chords
W = diaphragm width (ft),
G_a = apparent shear wall stiffness (k/in),
Δ_c = diaphragm chord slip (in),
x = distance from chord splice to nearest support (ft).

The three terms account for deflection due to diaphragm framing bending, shear, and chord slip,

$$\delta_{dia} = \frac{5vL^3}{8EAW} + \frac{0.25vL}{1000G_a} + \frac{\sum(x_i\Delta_c)}{2W}$$

respectively. This equation is similar to that of the code-accepted AWC SPDWS-2008 (AWC 2008) equation for deflection of diaphragms with wood sheathing on wood framing, which is given as

The difference in bending terms between the two equations stems from the fact that the SDPWS equation considers wood-sheathed, wood-framed diaphragms to act as a deep beam where only the outermost framing member acts as chords to resist the moment in the diaphragm (typically the double top plate). The equation of Pope et al. (2012) accounts for the contributions of all chords (purlins), not just the outer ones.

The third term, which accounts for chord slip, varies in how the cumulative distances from chord splices to the end walls are calculated because of how purlin splices are located in post-frame buildings. It is also assumed that the butt joints in the chords are not perfectly tight and that the slip of the tension chord equals the slip of the compression chord. Therefore, the total splice slip would be double that of the tension or compression slip alone (Pope et al. 2012). This explains why the third term in Pope's deflection equation is twice that of the SDPWS equation. The total eave deflection, Δ_{eave}, is the deflection of the shear wall added to the diaphragm deflection. The shear wall deflection can be calculated by Equation C.4.3.2 – 2 (AWC 2008):

$$\delta_{sh} = \frac{8vH_1^3}{EAb} + \frac{vH_1}{1000G_a} + \frac{H_1}{b}\Delta_a$$

E = modulus of elasticity of the end posts (psi)
A = cross-sectional area of the end wall posts (in^2)
b = shear wall length (ft)
Δ_a = shear wall anchorage slip (in)

Similar to the SDPWS equation for diaphragm deflection, the three terms of the shear wall deflection equation above account for deflection due to framing bending, shear, and wall anchorage slip, respectively. Because the posts are embedded in the ground for post-frame construction, it is assumed that no wall anchorage slip occurs, therefore eliminating the third term of the equation.

Embedded Post Foundation Design

Once the ground line moment is determined, the post embedment depth can be calculated using Equation 18 -3 in Section 1807.3.2.2 of the 2009 International Building Code (IBC). It should be noted that this equation applies to the situation where there is ground line constraint, such as provided by a concrete slab on grade. The posts on the leeward side of the building are commonly tied into the concrete slab with steel rebar.

$$d = \sqrt{\frac{4.25M_g}{S_3 b}}$$

d = post embedment depth (ft)
M_g = moment in the post at grade (ft-lb)
b = diameter of round pole or diagonal dimension of post (ft)

S_3 = allowable lateral soil-bearing pressure as set forth in Section 1806.2 based on a depth equal to the depth of embedment, psf. Section 1806.3.3 allows lateral pressures from Table 1806.2 to be increased by the tabular value for each additional foot of depth, to a maximum of 15 times the tabular value. Note that an initial guess of embedment is needed to calculate the starting value for S_3.

More extensive post embedment design equations to cover situations such as posts with collars and post uplift, as well as a variety of soil resistance assumptions, can be found in ANSI/ ASAE EP486.1 *Shallow Post Foundation Design* (ASAE 2005), McGuire (1998) and Meador (1997). For the simple constrained post case given in the IBC, the ASAE EP486.1 standard gives a more convenient form of the equation that does not require iterative calculations ($S' = S_3/d$)

$$d = \sqrt[3]{\frac{4.25M_g}{S'b}}$$

S' = allowable lateral soil-bearing pressure in psf/ft from Table 1806.2 (IBC 2009)

Table 2. Example Building Specifications

Width (Truss Length)	36 ft
Length (Along Slope)	60 ft
Height at Post Bearing	12 ft
Roof Slope	4/12 (18.43°)
Bay Spacing	10 ft
Number of Frames (Including End Walls)	7
Post Embedment Depth	4 ft
Post Grade And Species	No. 2 S. Pine
Post Size	6 x 6 Nominal
Roof Snow Load	30 psf
Roof Dead Load	5 psf
Concrete Slab?	Yes
Ceiling?	No
Wind Speed	80 mph
Exposure Category	B
Windward Wall, q_{ww}	8.13 psf
Leeward Wall, q_{lw}	-5.08 psf
Windward Roof, q_{wr}	3.05 psf
Leeward Roof, q_{lr}	-7.12 psf

*Negative loads act away from the surface in question. Positive loads

Design example

We will work the same example problem that is given in the NFBA'S *Post-Frame Building Design Manual,* Chapter 9. Table 2 gives the building specifications for the example.

Roof Diaphragm

Calculate maximum unit shear in roof diaphragm

$$v = \frac{K(q_{ww} - q_{LW})H_1L + (q_{WR} - q_{LR})H_2L}{2W}$$

Select roof diaphragm construction to carry <u>*at least 100.4 lb/ft.*</u>

Shear Wall

Assume a 12-ft wide overhead door in one end wall.

$V_{max} = v\,W = 100.4 * 36\text{ ft} = 3{,}614\text{ lb}$

$v_{shear\ wall} = V_{max} / (W_{bldg} - W_{opening})$ = 3,614 / (36 – 12) = 151 lb/ft

Select shear wall construction to carry <u>*at least 151 lb/ft.*</u>

Solve for Roof Diaphragm Chord Forces

The roof purlins need to be sized to carry all anticipated load combinations (e.g. dead, snow, live and wind). The axial chord forces can be calculated as follows.

Procedure

1. Solve for maximum shear force, V_{max}, in the roof diaphragm using the rigid roof equation.

V_{max} = 3,614 lb

2. Calculate the resulting uniform load, w, on the roof diaphragm, assuming the load is evenly distributed along the building length.

$w = 2V_{max}/L$ = 2* 3,614/60 = 120.5 lb/ft

3. Solve for the maximum bending moment, M, in the roof diaphragm, assuming a simple beam with pin and roller supports.

$M = wL^2/8$ = 120.5 (60)2/8 = 54,212 lb/ft

4. Solve for the maximum axial force on the perimeter purlin, T_n, using the following equation with (36/2 + 1) = 19 purlins.

From Table 1: $\alpha = 0.284$

$T_{max} = M\,\alpha / d = 54{,}212 * 0.284 / 36 = 428$ lb

The maximum chord force T_n is used to size the chord members and splices.

Post Moments

From the NFBA example:

E = 1.2 x 106 psi
I = 76.26 in4 (nominal 6x6 post)
Δ = 0.655 in
w = 8.13 psf * 10 ft / 12 in/ft = 6.78 lb/in

Maximum positive moment near $H_1/3$ down from top of post is

$$M^{+}_{\max} = \frac{9wH_1^2}{128} - \frac{9\Delta_{eave}EI}{8H_1^2}$$
$$= \frac{9(6.78)(144)^2}{128} - \frac{9(0.655)(1.2x10^6)(76.26)}{8(144)^2}$$
$$= \underline{6{,}633 in-lb}$$

The maximum negative moment at the ground line is

$$M^{-}_{\max} = \frac{wH_1^2}{8} + \frac{3\Delta_{eave}EI}{H_1^2}$$
$$= \frac{(6.78)(144)^2}{8} + \frac{3(0.655)(1.2x10^6)(76.26)}{(144)^2}$$
$$= \underline{26{,}246 in-lb}$$

Post Embedment

From ASAE EP486.1 Shallow Post Foundation Design:

$$d = \sqrt[3]{\frac{4.25M_b}{S'b}} = \sqrt[3]{\frac{4.25(26{,}246/12)}{(200)(0.648)}} = \underline{4.2ft}$$

How Conservative is the Simplified Lateral Design Approach?

Mill (2012) compared two options (fixed ground line support and pin-roller) of the simplified lateral design method with the more rigorous ANSI/ASAE EP484.2 (ASAE 2003) method over a wide range of building aspect ratios, diaphragm/shear wall stiffnesses, and wall heights. Space limitations do not permit us to show the details here, but Table 3 summarizes the predictions of unit shear, eave deflection and maximum post moment for the two methods divided by the predictions from ANSI/ ASAE EP484.2. In other words, a ratio of 1.0 means perfect agreement between the simplified and EP484.2 methods, while a ratio of 1.25 means that the simplified method conservatively overpredicts the value by 25%.

As expected, the simplified method gives more conservative predictions of unit shear and post moment as the building length increases relative to its width. When the ground line is modeled as a pin (at 0.7 times the post embedment depth) and roller at the ground line, the predictions of post moment are closer to the EP484.2 method. For buildings with an aspect ratio up to 3, the conservative estimates of unit shear and post moment may still give economic designs with regard to post selection and embedment depth. Of course, a design professional can always apply a more rigorous design approach that accounts for diaphragm/frame interaction to gain some efficiency as justified.

Summary and Conclusions

There is a need to educate design and building regulatory professionals about lateral design of post-frame buildings. The objective of this article is to present a streamlined approach that is easier to learn than the more rigorous methods (that account for diaphragm/frame interaction) and will yield conservative designs. The method presented conservatively ignores the contribution of the frames in resisting lateral loads. For buildings with length-to-width ratios of 3 and less, predictions of unit shears and post moments are relatively close to those predicted with the more rigorous method, because of the fact that roof diaphragms are so much stiffer than the frames. Depending on the needs of the design professional, the added investment of time to learn the more rigorous methods may be justified.

Table 3. Comparison of Unit Shear, Deflection and Post Moment to ANSI/ASAE EP484.2.

L:W	hw =16 ft					
	Simplified—Fixed			Simplified—Pin/Roller		
	v Ratio	Δ_{eave} Ratio	*M* Ratio	*v* Ratio	Δ_{eave} Ratio	*M* Ratio
1	0.97	1.05	1.17	1.02	1.10	1.00
2	1.02	1.08	1.19	1.07	1.14	1.02
3	1.09	1.18	1.25	1.15	1.24	1.08
4	1.18	1.31	1.34	1.24	1.37	1.18

Note

1. Eave deflection can be calculated as the sum of the shear wall and roof diaphragm deflections. Equations for calculating deflections for conventional wood-frame diaphragms are given in the 2009 IBC (International Code Council, 2009) and in ANSI/AF&PA SDPWS-2008 (AF&PA, 2008). A modification of the diaphragm deflection equation for metal-clad post frame that includes load sharing of purlins and slip at purlin connections is given in this paper.

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